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Report No. UT-98.17A

LIQUEFACTION HAZARD EVALUATION OF INTERSTATE, FEDERAL, AND STATE HIGHWAY BRIDGE SITES IN UTAH: EXECUTIVE SUMMARY & IMPLEMENTATION PLAN

Prepared For:

Utah Department of Transportation
Research Division

Submitted By:

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December 30, 1998

UDOT RESEARCH DIVISION REPORT ABSTRACT

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16. Abstract

The purpose of this study is to evaluate the liquefaction hazard at bridge sites throughout the state of Utah and to test and suggest revisions to the screening guide developed by Youd (1998) for the National Center for Earthquake Engineering Research (NCEER). The procedure involved regional screening of liquefaction potential, site-specific evaluations of liquefaction hazard, and calculation of liquefaction-induced ground failures

The final objective of the study is to prioritize the bridge sites in Utah for future investigations. Sites are prioritized according to the presence of liquefiable sediment and the quality and availability of subsurface data. Sites classed as Priority I are located at river crossings and either have a confirmed presence of liquefiable sediment or insufficient information to evaluate liquefaction. Priority II sites have a confirmed presence of liquefiable sediment, but are not located at river crossings. Sites classed as Priority III are not located at river crossings and have insufficient data available to evaluate susceptibility. Priority IV sites have a confirmed low liquefaction hazard.

A principal emphasis of this study was evaluation of the I-15 corridor. The results show that nearly all of the bridge sites in Salt Lake County are underlain by possibly liquefiable sediment. An example calculation of liquefaction-induced ground failure hazard at the 600 South off ramp evaluated embankment stability and deformation, ground displacement due to lateral spread, ground settlement, and bearing capacity. The ground failure analyses showed that damaging ground displacements are not likely to occur at the 600 South site, even though liquefaction might occur.

A few hundred bridge sites were reviewed by regional screening procedures and 325 bridge sites were analyzed using the site-specific screening procedures. Of these sites, 279 were identified as underlain by possibly liquefiable soil layers. Twenty-five of these sites were identified as Priority I sites for further investigation (table 7). The Priority I sites are at river crossings, a setting in which most past bridge damage due to liquefaction has occurred. The remaining 254 bridge sites possibly underlain by liquefiable sediment were classed as Priority II sites. These sites are tabulated in appendix A of the full project report.

An implementation program is proposed with two phases of investigation. Phase I should study the Priority II sites along the I-15 corridor because of their importance and the opportunity to evaluate liquefaction hazards at little additional cost during the planned reconstruction program. Phase II would investigate and remediate Priority I sites. The possible liquefaction hazard at Priority II and Priority III sites is sufficiently low that a special program may not be required to investigate the hazard at these sites. Some critical structures, such as major interchanges or bridges in essential emergency transportation routes, should be scheduled for investigation with or immediately following the Phase II investigation. Otherwise, the hazard should be investigated whenever opportunity arises, such as when bridge upgrades or replacements are planned.

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DISCLAIMER

UNIT CONVERSIONS

Acceleration	9.81 m/s^2 = 386.22 $in./s^2$ = 32.185 ft/s^2, Paris: g = 9.80665 m/s^2 London: g = 3.2174 x 10^1 ft/s^2
Area	1 m^2 = 1.5500 x 10^3 $in.^2$ = 1.0764 x 10^1 ft^2 = 1.196 yd^2 = 10^6 mm^2 = 10^4 cm^2 = 2.471 x 10^{-4} $acres^2$ = 3.861 x 10^{-7} mi^2
Coefficient of Consolidation	1 m^2/s = 10^4 cm^2/s = 6 x 10^5 cm^2/min = 3.6 x 10^7 cm^2/h = 8.64 x 10^8 cm^2/day = 2.628 x 10^{10} $cm^2/month$ = 3.1536 x 10^{11} $cm^2/year$ = 1.550 x 10^3 $in.^2/s$ = 4.0734 x 10^9 $in.^2/month$ = 1.3392 x 10^8 $in.^2/day$ = 4.8881 x 10^{10} $in.^2/year$ = 9.4783 x 10^5 ft^2/day = 2.8830 x 10^7 $ft^2/month$ = 3.3945 x 10^8 $ft^2/year$
Flow	1 m^3/s = 10^6 cm^3/s = 8.64 x 10^4 m^3/day = 8.64 x 10^{10} cm^3/day = 3.5314 x 10^1 ft^3/s = 3.0511 x 10^6 ft^3/day
Force	10 kN = 2.2482 x 10^3 lb = 2.2482 kip = 1.1241 t (short ton = 2000 lb) = 1.0194 x 10^3 kg = 1.0194 x 10^6 g = 1.0194 T (metric ton = 1000 kg) = 10^9 dynes = 3.5971 x 10^4 ounces = 1.022 tl (long ton = 2200 lb)
Force per Unit Length	1 kN/m = 6.8526 x 10^1 lb/ft = 6.8526 x 10^{-2} kip/ft = 3.4263 x 10^{-2} t/ft = 1.0194 x 10^2 kg/m = 1.0194 x 10^{-1} T/m
Length	1 m = 3.9370 x 10^1 in. = 3.2808 ft = 1.0936 yd = 10^{10} Amgstrom = 10^6 microns = 10^3 mm = 10^2 cm = 10^{-3} km = 6.2137 x 10^{-4} mile = 5.3996 x 10^{-4} nautical mile
Moment or Energy	1 kN.m = 7.3759 x 10^2 lb.ft = 7.3759 x 10^{-1} kip.ft = 3.6879 x 10^{-1} t.ft = 1.0194 x 10^3 g.cm = 1.0194 x 10^2 kg.m = 1.0194 x 10^{-1} T.m = 10^3 N.m = 10^3 Joule
Moment of Inertia	1 m^4 = 2.4025 x 10^6 $in.^4$ = 1.1586 x 10^2 ft^4 = 6.9911 x 10^{-1} yd^4 = 10^8 cm^4 = 10^{12} mm^4
Moment per Unit Length	1 kN.m/m = 2.2482 x 10^2 lb.ft/ft = 2.2482 x 10^{-1} kip.ft/ft = 1.1241 x 10^{-1} t.ft/ft = 1.0194 x 10^2 kg.m/m = 1.0194 x 10^{-1} T.m/m
Pressure	100 kPa = 10^2 kN/m^2 = 1.4503 x 10^1 $lb/in.^2$ = 2.0885 x 10^3 lb/ft^2 = 1.4503 x 10^{-2} $kip/in.^2$ = 2.0885 kip/ft^2 = 1.0442 t/ft^2 = 7.5003 x 10^1 cm of Hg (0 °C) = 1.0197 kg/cm^2 = 1.0197 x 10^1 T/m^2 = 9.8689 x 10^{-1} Atm = 3.3455 x 10^1 ft of H_20 (4 °C) = 1.0000 bar = 10^6 $dynes/cm^2$
Temperature	°C = 5/9 (°F - 32), °K = °C + 273.15
Time	1 yr. = 12 mo. = 365 day = 8760 hr = 5.256 x 10^5 min = 3.1536 x 10^7 s
Unit Weight, Coefficient of Subgrade Reaction	10 kN/m^3 = 6.3654 x 10^1 lb/ft^3 = 3.6837 x 10^{-2} $lb/in.^3$ = 1.0196 g/cm^3 = 1.0196 T/m^3 = 1.0196 x 10^3 kg/m^3
Velocity or Permeability	1 m/s = 3.6 km/h = 2.2369 mile/h = 6 x 10^1 m/min = 10^2 cm/s = 1.9685 x 10^2 ft/min = 3.2808 ft/s = 1.0346 x 10^8 ft/year = 2.8346 x 10^5 ft/day
Volume	1 m^3 = 6.1024 x 10^4 $in.^3$ = 3.5315 x 10^1 ft^3 = 7.6455 10^1 yd^3 = 10^9 mm^3 = 10^6 cm^3 = 10^3 dm^3 = 10^3 liter = 2.1998 x 10^2 gallon (U.K.) = 2.6417 x 10^2 gallon (U.S.)
Volume Loss in a Tubing	1 $cm^3/m/kPa$ = 8.91 x 10^{-4} $in.^3/ft/psf$

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Introduction

The objective of this study is to evaluate the liquefaction hazard at bridge sites in Utah and to test and revise a screening guide being developed for the National Center for Earthquake Research (NCEER) under the sponsorship of the Federal Highway Administration (Youd, 1998). The guide is to be made available for liquefaction hazard screening to other states and agencies. Highway segments evaluated in this study are highlighted on the map in figure 1.

Liquefaction has been a major cause of earthquake damage to bridges during past earthquakes. For example, 266 highway and railway bridges were damaged to varying degrees by liquefaction-induced lateral spread during the 1964 Alaska earthquake (Youd, 1993). Many of these bridges were damaged beyond repair and several collapsed. Nearly all of the damage occurred at river crossings where lateral spread of flood plain deposits caused abutments and piers to shift laterally toward river channels, buckling superstructures or thrusting them laterally into or over abutments. Similar damage has been reported following several other large earthquakes (Youd, 1993). In the event of a large earthquake in Utah, bridges could suffer similar damage. Much of this damage could be avoided through identification of hazardous bridge sites and implementation of mitigative measures. This study provides a first step in such a mitigative effort by identifying bridge sites possibly underlain by liquefiable sediment.

The screening guide provides a systematic application of standard criteria for assessing liquefaction hazard and for prioritizing sites for further investigation. The guide proceeds from simple low-cost evaluations, requiring little site-specific data, to more complex, time consuming, and rigorous analyses. The screening guide uses existing information, such as past liquefaction hazard analyses, geologic maps, foundation investigation reports, and does not require development of new site data. By application of the simpler evaluations first, sites in low hazard areas may be classified as low hazard with minimal time and effort. Only bridge sites in the more vulnerable settings need to be analyzed with the more rigorous and time-consuming site-specific evaluations.

At each step in the analysis, a conservative assessment of hazard is made. Where there is clear evidence that liquefaction or damaging ground displacements are unlikely, the site is classified as low liquefaction hazard and low priority for further investigation (Priority IV). At that point, the evaluation is complete for that bridge site. If there is evidence that a liquefaction hazard may exist, the site is classed as possibly liquefiable and the analysis proceeds to the next step. The evaluation proceeds step by step until the bridge site is classified as either hazardous, nonhazardous or insufficient information to fully evaluate the hazard. If the available information is inadequate, geologic, hydrologic, topographic, and the importance of the structure information are used to prioritize the site for further investigation. The final outcome of the screening is an assignment of each bridge site to one of the following four priorities for further investigation and possible mitigation:

Priority I sites: Bridge sites with the highest priority for further investigation and possible mitigation are categorized as Priority I sites. Those sites are likely to be underlain by liquefiable sediment or sensitive clay that could induce damaging ground or foundation

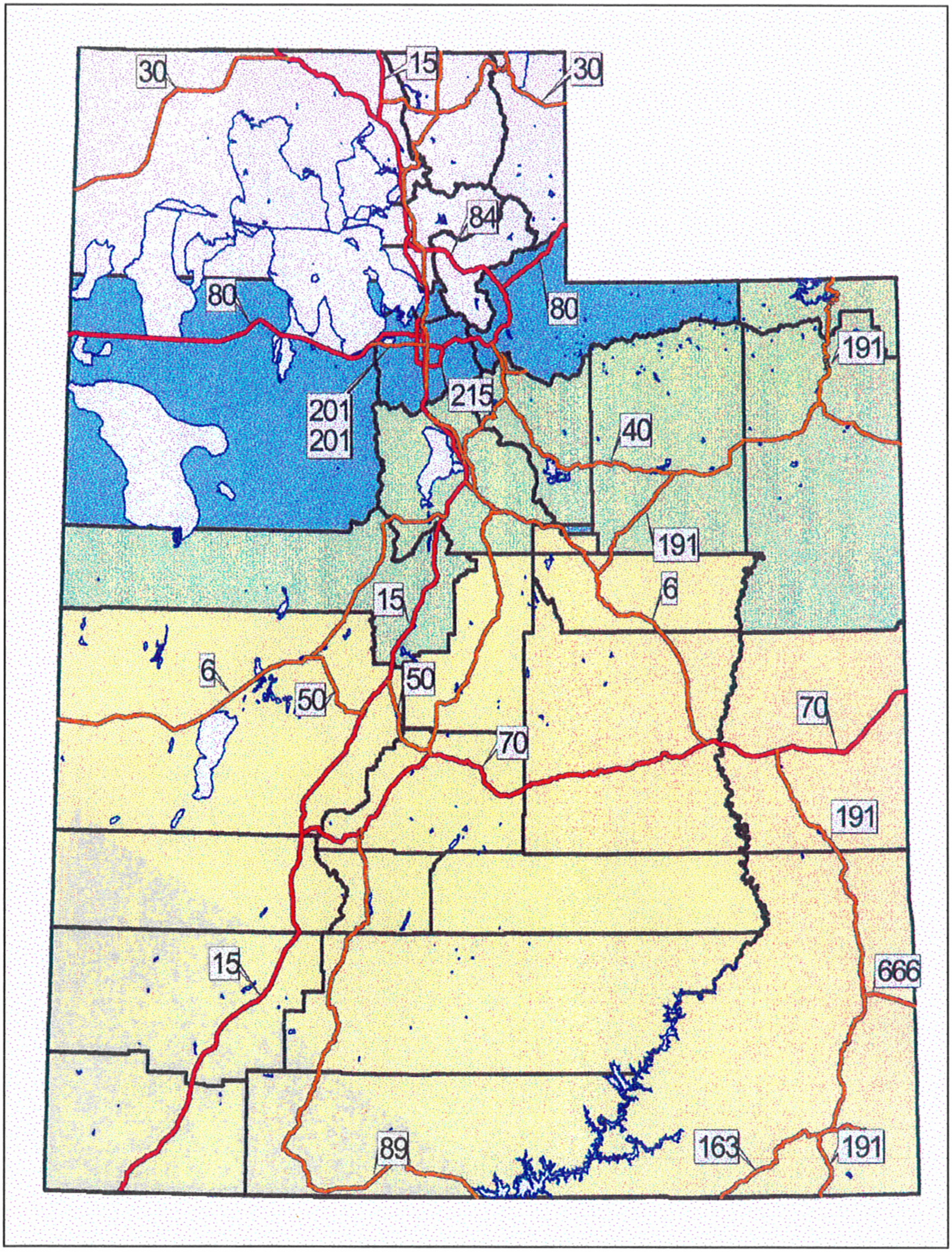

Figure 1: **Highways in Utah for which liquefaction hazard at bridge sites was evaluated**

displacements during an earthquake. Liquefiable sediment at these sites was either confirmed or the available information was insufficient to eliminate the possibility of liquefiable sediment. A second criterion is that the site is located in an area highly vulnerable to ground failure, such as river crossings, near lakes or other bodies of water, near a steep slope, or approached by thick embankments (greater than 5 m) overlying possibly liquefiable sediment.

Priority II sites: Bridge sites with the second highest priority for investigation and possible mitigation are localities confirmed to be underlain by liquefiable sediments or sensitive clay, but where the available site information is insufficient to fully evaluate ground failure or foundation stability hazards. Priority II sites are located away from rivers, other bodies of water, steep slopes or thick embankments (otherwise they would be classed as Priority I sites). Liquefaction at these sites could cause ground settlement and possible foundation instability, but damaging lateral ground displacements are unlikely. These sites are assigned a moderate priority for further investigation.

Priority III sites: Bridge sites with third priority for further investigation are those with insufficient available geotechnical information to fully assess the liquefaction hazard. These sites are also located away from rivers, other bodies of water, steep slopes or thick embankments (otherwise they would be classed as Priority I sites). Liquefaction at these sites could cause ground settlement and possible foundation instability, but damaging lateral ground displacements are unlikely. These sites are assigned a moderate to low priority for further investigation.

Priority IV: Bridge sites with the lowest priority for further investigation are those where the screening evaluation indicated very low liquefaction susceptibility or ground failure hazard. These bridges were assigned a low priority for further investigation or mitigation.

The general screening procedure, as outlined in the flow chart reproduced in figure 2, consists of three sequential levels of evaluation: regional screening, site-specific evaluation, and assessment of site and foundation stability. Each succeeding level of investigation requires more detailed information and more complex and more time consuming analyses. Each level contains several intermediate steps with a decision required at the end of each step. If the decision is that liquefaction or ground failure potential is very low, the site is classed as low liquefaction hazard and low priority for further investigation (Priority IV), and the evaluation is complete for that site. If the conclusion is that a possible hazard exists, the evaluation continues to the next step.

Regional Screening

Regional screening involves assessment of liquefaction potential from the general setting of the bridge site rather than from site-specific information. As noted in the introduction and on figure 2, regional evaluation involves the following steps:

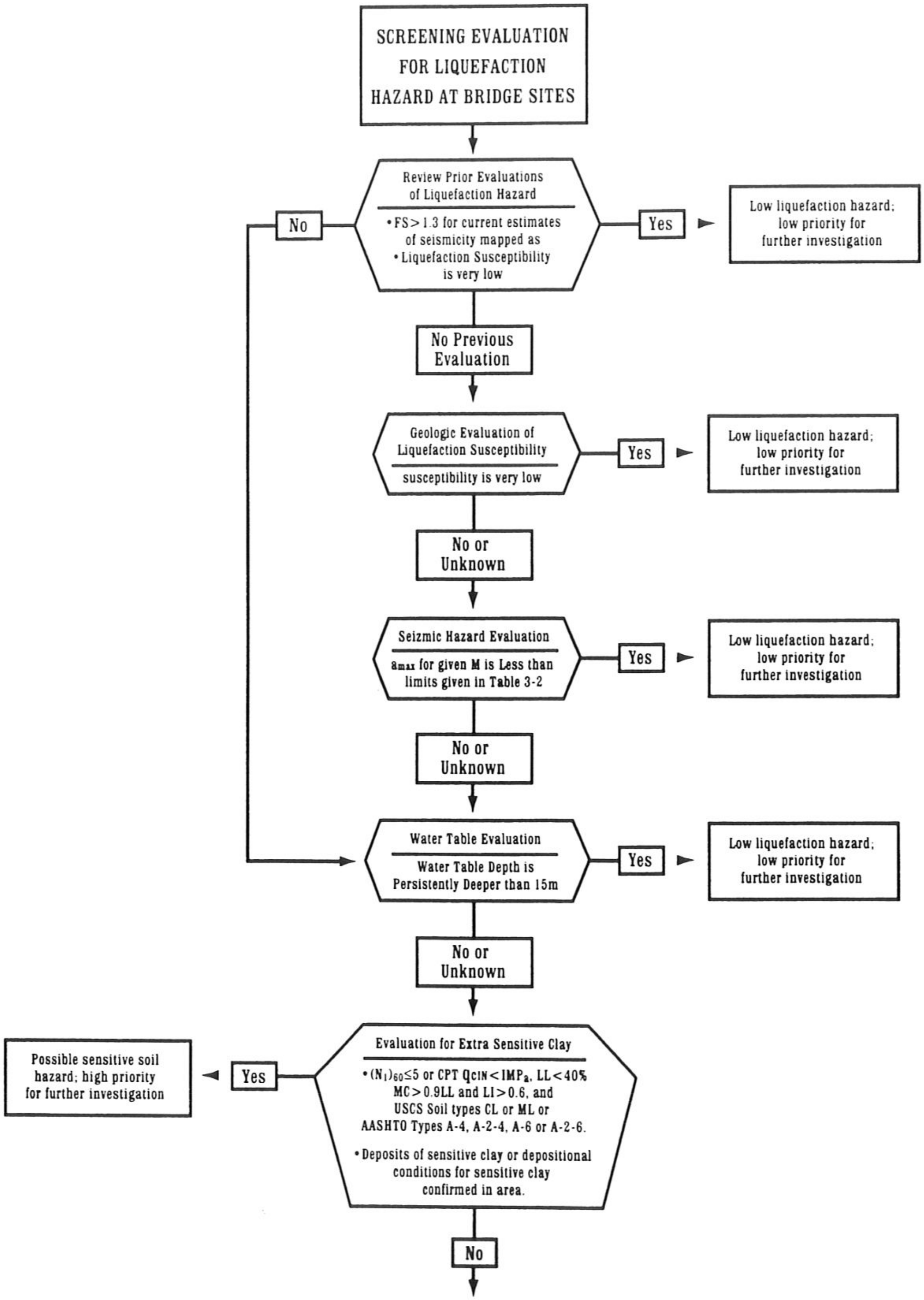

Figure 2: Flow diagram showing steps and criteria for screening of liquefaction hazard for highway bridges-Part I

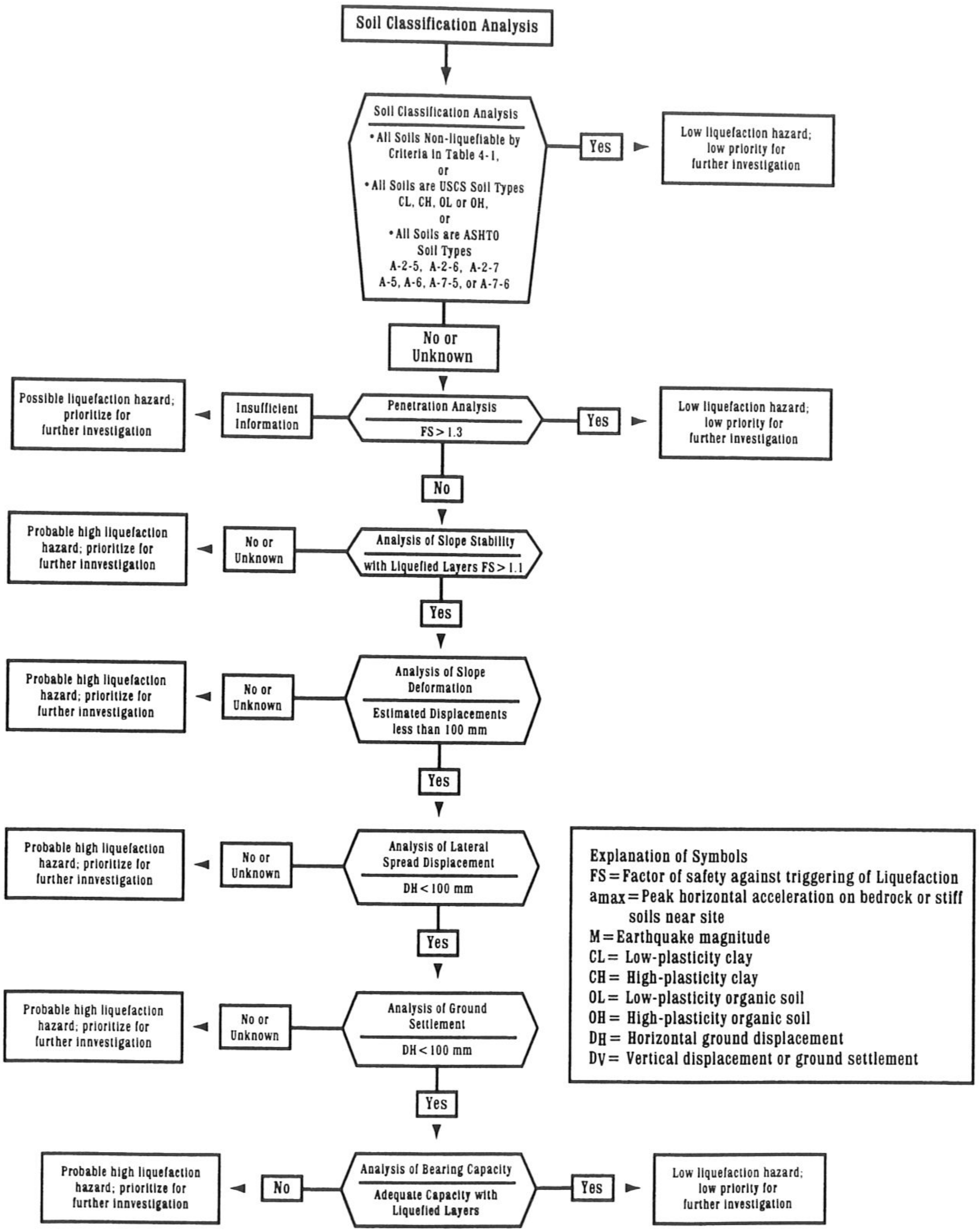

Figure 2 (Continued): Flow diagram showing steps and criteria for screening of liquefaction hazard for highway bridges-Part II

Prior Evaluation of Liquefaction Hazard

As a first step, available information on liquefaction susceptibility at or near the site should be evaluated. Such information might include: (1) postearthquake reconnaissance reports that indicate whether liquefaction effects, such as sand boils and ground fissures, were reported at or near the bridge site following past earthquakes in the region; (2) liquefaction potential or hazard maps for the region in which the site is located; and (3) previous site-specific liquefaction hazard evaluations for the bridge. Depending on past earthquake activity and hazard studies in the area, excellent to little information may be available for screening the liquefaction hazard.

Postearthquake Investigations

Because liquefaction tends to recur at the same site during successive earthquakes, past occurrences of liquefaction are rather certain indicators of high liquefaction susceptibility, providing site conditions, such as water table depth, have not changed. In Utah, minor effects of liquefaction were observed following the 1934 Hansel Valley, the 1962 Cache Valley, and the 1992 St. George earthquakes. Only the 1992 St. George earthquake generated liquefaction effects near bridge structures. Investigations following that earthquake identified sand boils and minor ground fissures near the I-15 crossing over the Virgin River (Black et al., 1995). Based on that history and a lack of sufficient information to fully assess the ground failure hazard, that site was assigned to the Priority I classification, indicating a high priority for further investigation and possible application of mitigative measures.

Liquefaction Hazard Maps

Liquefaction potential maps have been compiled for all of the counties along the Wasatch Front and for several areas in central Utah (Anderson and others, 1994a,b,c,d,e). For example, a segment of the liquefaction potential map for Utah County is reproduced in figure 3. In addition, Mabey and Youd (1989) compiled probabilistic liquefaction severity index (LSI) maps for the State (figure 4). The LSI map provides estimates of maximum lateral spread displacements that are likely to occur at sites underlain by liquefiable sediments. These maps provided hazard information for the regional screening evaluations. Bridge sites located within areas delineated as very low liquefaction potential or low LSI potential (LSI less than 5 with 2 percent probability of being exceeded in 50 years) were assigned a Priority IV classification--low liquefaction hazard and low priority for further investigation-- and the evaluation was complete for those sites. Several important sites so classified were further evaluated, however, to confirm the low hazard rating. Bridge sites characterized with LSI greater than 5 and moderate or higher liquefaction potential were further evaluated using site-specific analyses.

Implementation of this evaluation is illustrated for the northern part of Utah County. No historical accounts of liquefaction have been reported from that area, so a review was made of liquefaction hazard maps for the area. Mabey and Youd (1989) characterize that area with an LSI greater than 30 with 2 percent probability of being exceeded in 50 years (figure 4). The liquefaction potential maps by Anderson and others (1994c), reproduced in figure 3, confirm this hazard. Starting at the north end of the county, I-15 passes over an upland ridge called Point of

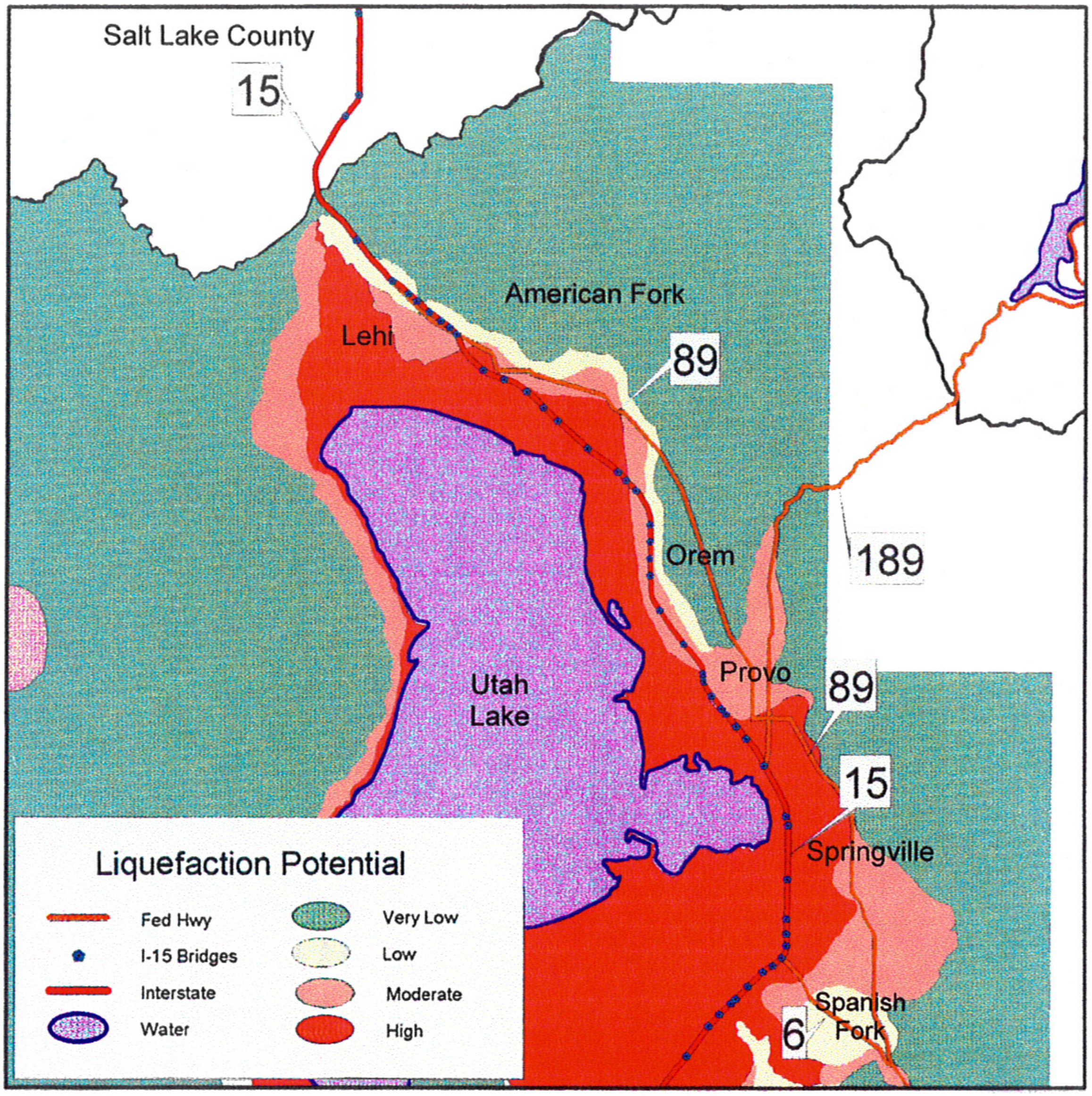

Figure 3: Liquefaction potential map for Utah County showing routes of I-15 and other major highways

the Mountain, an area zoned with very low liquefaction potential. Six bridge sites located in the very-low zone (between the Salt Lake County line and Lehi) were classified as Priority IV-low liquefaction hazard and low priority for future study. From Lehi to Orem, I-15 traverses a narrow zone of moderate and a broader zone of high liquefaction potential (figure 3). Three I-15 bridges lie in the moderate potential zone and eight bridges lie in the high potential zone. These eleven bridge sites were classed as having a possible liquefaction hazard at this juncture and were further evaluated with site specific procedures, as indicated on the flow chart (figure 2).

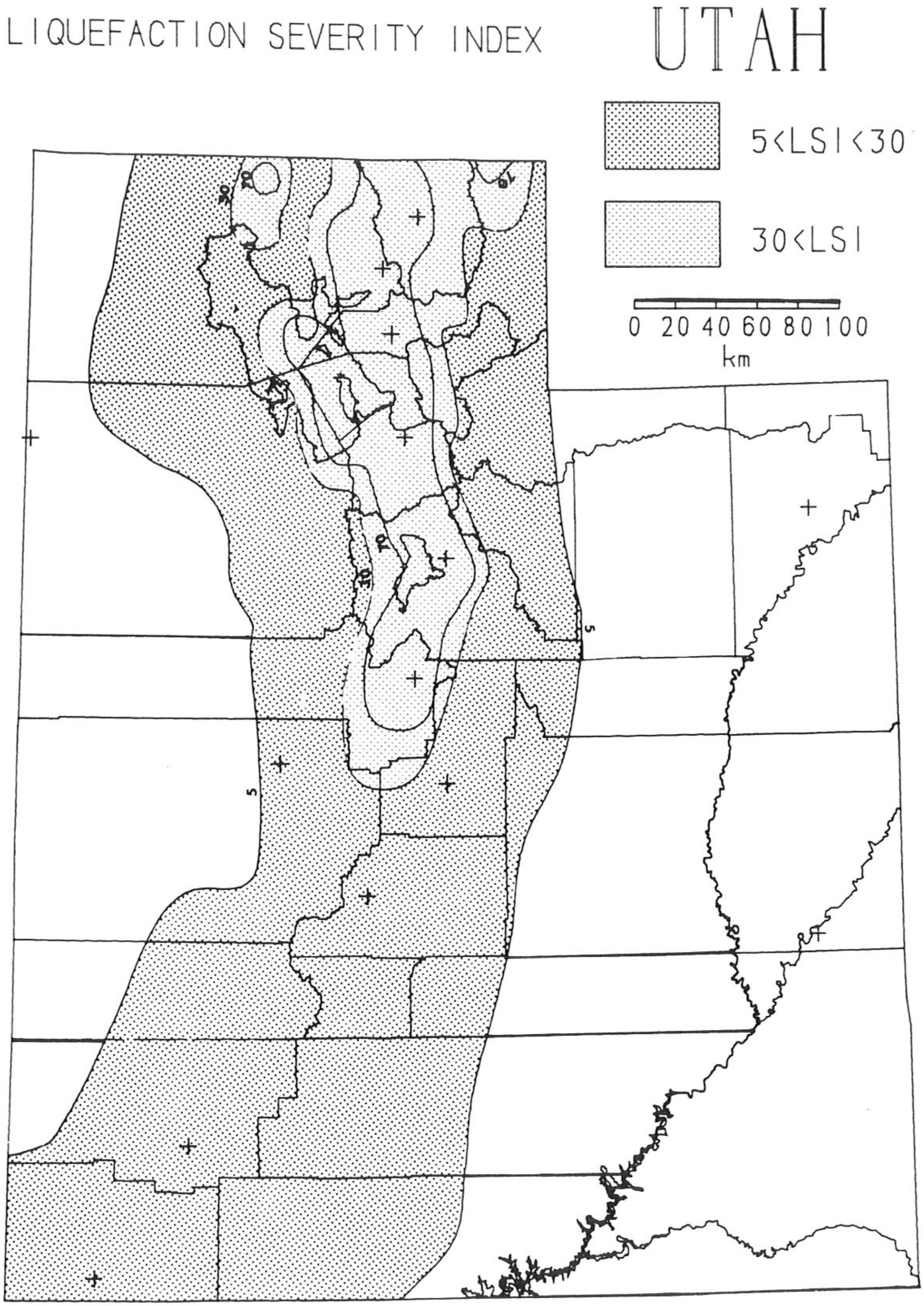

Figure 4: Liquefaction severity index map for Utah (after Mabey and Youd, 1989)

Geologic Evaluation

For areas where liquefaction hazard maps have not been compiled, geologic evaluations were used to assess liquefaction susceptibility. Liquefiable sediments are not randomly distributed in natural terrain, but rather develop only within particularly favorable geologic environments. The criteria listed by Youd and Perkins (1978) in table 1 are commonly used for geologic screening. Quaternary geologic maps are required for this evaluation. Where Quaternary geologic maps are not available, a local map may be compiled from a review of geologic literature, aerial photo interpretation or site reconnaissance visits. Quaternary maps have been compiled for most parts of Utah. If the geologic maps indicate nonliquefiable sediments types (table 1), such as pre-Pleistocene sedimentary or rock units, the site was assessed as low hazard and low priority for further investigation (Priority IV).

Highway routing in much of central and southern Utah cross geologic units that are too old or too indurated to liquefy. For example, figure 5 is a geologic map for part of south-central Utah with the route of I-70 marked between Cove Fort and Salina. Nearly all of the bridge sites in this region are underlain by sediment or bedrock units that are too old, dense or lithified to liquefy. Bridge sites in this area were classified as Priority IV with low priority for further investigation.

Seismic Hazard Evaluation

A certain threshold of seismic energy must propagate through a site during seismic shaking to generate a liquefied condition. That threshold is a function of the density of the material, depth of sediment burial, and several other factors. Even the most susceptible natural materials require a finite amount of seismic energy to generate a liquefied condition. Thus, seismic energy from small local earthquakes or larger, distant events may be insufficient to generate liquefaction. The seismic factors most commonly used by geotechnical engineers to characterize seismic energy propagating through a site are earthquake magnitude (M) and peak horizontal acceleration at the ground surface (a_{max}). For this screening evaluation, the criteria listed in table 2 were applied to identify sites in Utah with sufficiently low shaking potential where the risk of liquefaction is acceptably small. Magnitude and maximum acceleration for an area were compared with the values listed in table 2. If a_{max} is below the value listed for the given magnitude, the area, and all bridge sites within the area, may be safely classed as low liquefaction hazard and low priority for further investigation.

For liquefaction hazard screening in much of eastern Utah, where earthquake potential is generally low, estimates of a_{max} were taken from probabilistic maps prepared by the US Geological Survey (USGS). To be conservative, values of a_{max} with 10 percent or less probability of exceedance in 250 years (average return period of about 2,500 years) were used. To be consistent with these a_{max} values, the largest magnitudes of earthquakes expected in any 2,500 year period (0.0004 annual probability of occurrence) per 1,000 km^2 were also used. Figure 6 shows a_{max} values with 10 percent probability of exceedance in 250 years that have been delineated for Utah by the US Geological Survey. These maps are continually being revised and updated; for future studies, the most recent map should be used. Similarly, a map showing magnitudes of

Table 1: Estimated susceptibility of sedimentary deposits to liquefaction during strong seismic shaking (modified from Youd and Perkins, 1978).

Type of deposit	General distribution of cohesionless sediments in deposits	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by age of deposit)			
		< 500 yr	Holocene	Pleistocene	Pre-Pleistocene
(1)	(2)	(3)	(4)	(5)	(6)
(a) Continental Deposits					
River channel	Locally variable	Very High	High	Low	Very Low
Flood plain	Locally variable	High	Moderate	Low	Very Low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very Low
Marine terraces and plains	Widespread	-	Low	Very Low	Very Low
Delta and fan-delta	Widespread	High	Moderate	Low	Very Low
Lacustrine and playa	Variable	High	Moderate	Low	Very Low
Colluvium	Variable	High	Moderate	Low	Very Low
Talus	Widespread	Low	Low	Very Low	Very Low
Dunes	Widespread	High	Moderate	Low	Very Low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very Low	Very Low
Tuff	Rare	Low	Low	Very Low	Very Low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very Low	Very Low
Sebka	Locally variable	High	Moderate	Low	Very Low
(b) Coastal Zone					
Delta	Widespread	Very High	High	Low	Very Low
Esturine	Locally variable	High	Moderate	Low	Very Low
Beach					
High wave energy	Widespread	Moderate	Low	Very Low	Very Low
Low wave energy	Widespread	High	Moderate	Low	Very Low
Lagoonal	Locally variable	High	Moderate	Low	Very Low
Fore shore	Locally variable	High	Moderate	Low	Very Low
(c) Artificial Fill					
Uncompacted fill	Variable	Very High	-	-	-
Compacted fill	Variable	Low	-	-	-

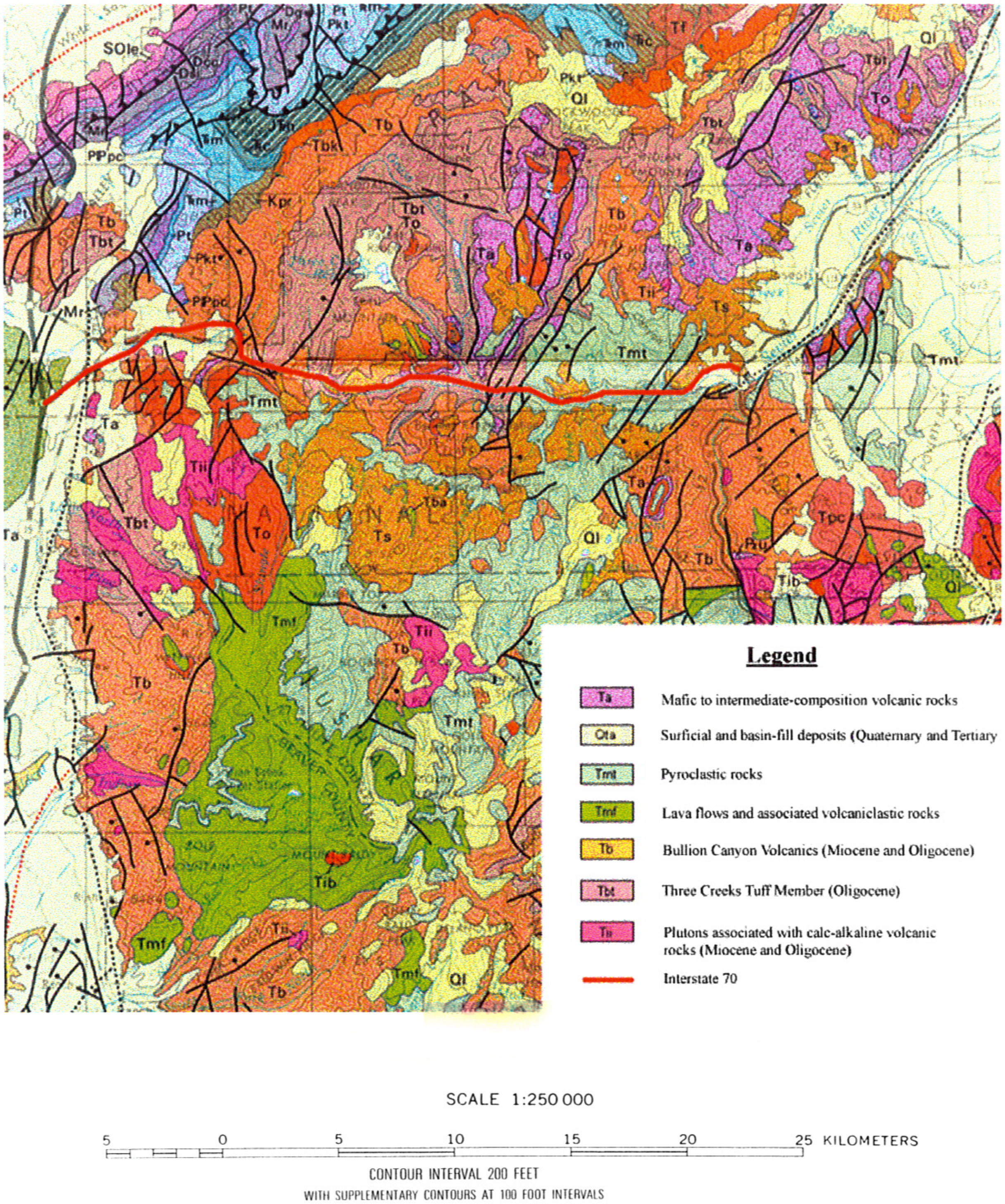

Figure 5: Geologic Map of the Richfield, Utah Area Showing Route of I-70 through nonliquefiable pre-Pleistocene geologic formations

Figure 6: Probabilistic peak horizontal acceleration with a 10% chance of exceedence in 250 years for Utah and portions of the Western U.S. (Algermissen et. al., 1990)

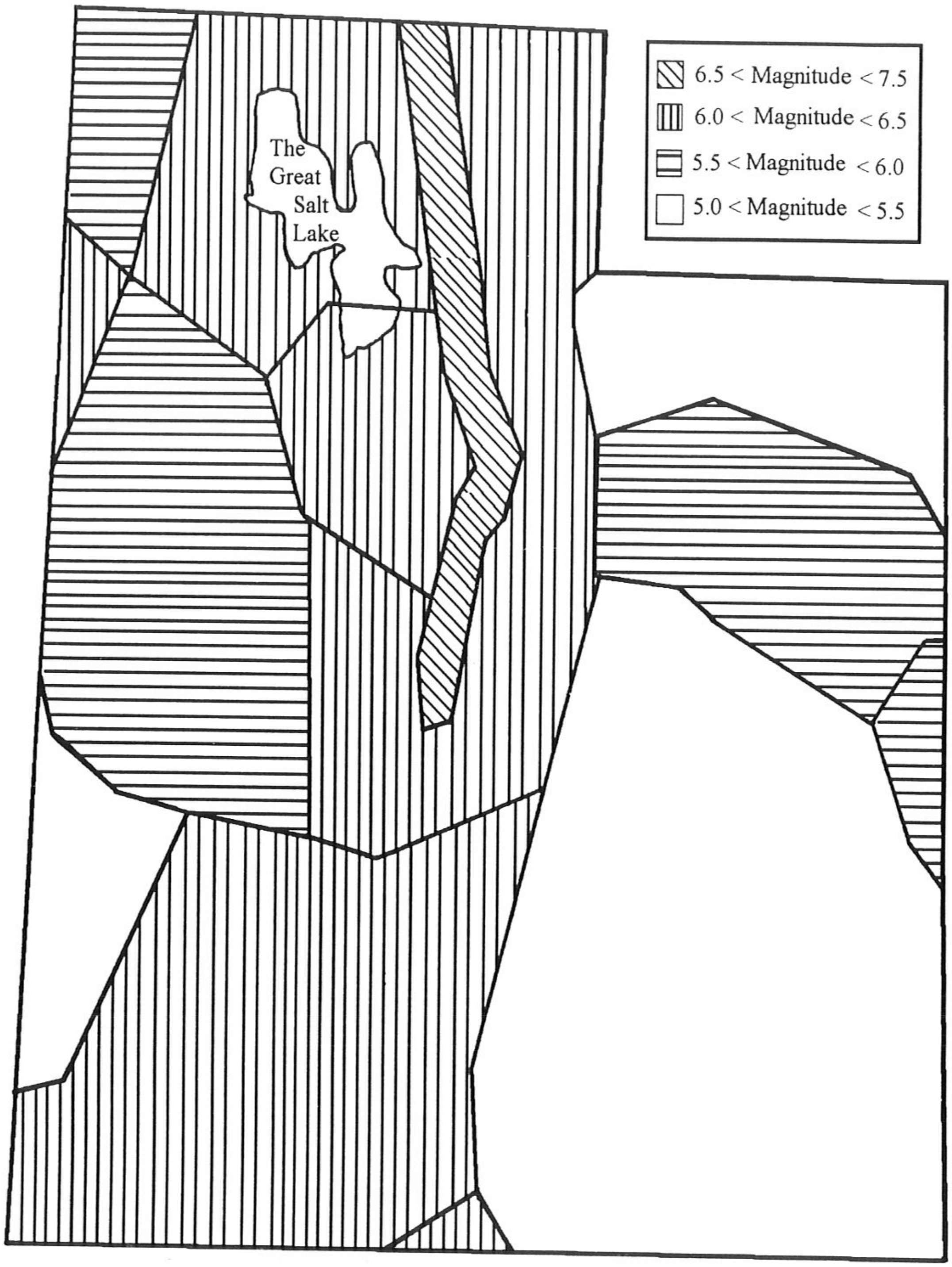

Figure 7. Map of Utah showing seismic source zones and maximum earthquake magnitudes with <0.0004 annual rate of occurrence per 1,000 km^2 (>2,500-year recurrence interval) (data from Hanson and Perkins, 1995)

Table 2: Minimum earthquake magnitudes and peak horizontal ground accelerations, with allowance for local site amplification, that are capable of generating liquefaction in very susceptible natural deposits (after Youd, 1998).

Earthquake Magnitude, M_w	Liquefaction Hazard for Bridge Sites	
	Soil Profile Types I and II (stiff sites)	Soil Profile Type III (soft sites)
$M < 5.2$	Very low hazard for $a_{max} < 0.4g$	Very low hazard for $a_{max} < 0.1g$
$5.2 < M < 6.4$	Very low hazard for $a_{max} < 0.1g$	Very low hazard for $a_{max} < 0.05g$
$6.4 < M < 7.6$	Very low hazard for $a_{max} < 0.05g$	Very low hazard for $a_{max} < 0.025g$
$M > 7.6$	Very low hazard for $a_{max} < 0.025g$	Very low hazard for $a_{max} < 0.025g$

earthquakes in Utah with annual frequencies of occurrence of 0.0004 per year per 1,000 km^2 is reproduced in figure 7.

An example of how this seismic evaluation was implemented in Utah is illustrated for the Interstate and Federal highways east of the 111° meridian. That area is traversed by all or parts of Highways I-70, 6, 40, 50, 163, 191, and 666. That region is characterized by earthquake magnitudes less than 6.0 for the given probability of occurrence. Most of the area in the southeast and northeast sectors of the region are characterized by earthquakes with magnitudes less than 5.5. Similarly, peak accelerations in the area with 10 percent probability of occurrence in 250 years decline from near 0.25 g along the eastern margin of the zone to less than 0.05 g in the southeastern part of the State. Due to the arid climate and generally shallow soils, soil profile types in that part of the State are almost exclusively Types I and II (stiff sites). Applying the criteria in table 2 and the peak acceleration and magnitude values from figures 6 and 7, respectively, indicates that all bridge sites in this region are characterized by low liquefaction hazard and were classified as Priority IV sites--low priority for further investigation.

These same criteria were also applied to bridge sites in southwestern Tooele, and western Juab, Millard, Beaver and Iron Counties where expected earthquake magnitudes are less than 6 and peak accelerations are less than 0.25 g. All bridges in this region were classified as Priority IV sites.

Water Table Evaluation

Because liquefaction occurs only in saturated or very nearly saturated soils, sediments above the unconfined ground water table, permanent or perched, are generally immune to liquefaction. Also, soils generally become older, denser, and more liquefaction resistant with depth. As a consequence, most episodes of liquefaction have occurred at depths shallower than 10 m and few episodes have occurred at depths greater than 15 m in natural soils. For most bridge sites, liquefaction at depths greater than 10 m would not adversely affect site stability.

Table 3: Relative liquefaction susceptibility as a function of groundwater table depth (after Youd, 1998)

Groundwater Table Depth	Liquefaction Susceptibility
< 3 m	Very High
3 m to 6 m	High
6 m to 10 m	Moderate
10 m to 15 m	Low
> 15 m	Very Low

Based on these relationships, liquefaction susceptibility for bridge sites is summarized in table 3 (Youd, 1998). For bridges over dry creeks the critical depth should be the free ground water level beneath the stream bed.

These criteria were generally applied for liquefaction hazard screening in Utah. Transient rises of ground water level during occasional floods were ignored for this study because of the small percentage of time in which the shallower sediments are saturated. Two principal sources of groundwater data were used for the screening evaluation. The principal source was shallow ground water maps by Hecker et al. (1988). The second source was water table measurements taken from borehole logs cataloged in bridge foundation investigation reports. The map by Hecker et al. delineated areas with groundwater depths less than or greater than 9 m. In order to directly use these maps, low liquefaction hazard was defined for depths greater than 9 m rather 10 m as listed in table 3. That slight difference in the ground water depth criterion should not significantly affect the hazard for arid areas such as Utah. For hazard mapping outside the Wasatch front, where seismic demand is low as indicated by figures 6 and 7, bridge sites with free ground water levels deeper than 9 m were classified as low hazard or Priority IV sites.

As an example, a water table depth greater than 9 m is indicated by Hecker et al. (1988) for the I-15 bridge over the "I" line (station 711+80) between Fillmore, Millard County, and the Juab County line. The "I" line bridge is in an area of unconsolidated sediments several meters thick overlying conglomeratic bedrock. Based on this information, the bridge was classified as a Priority IV site. Nevertheless, borehole data from the site were analyzed to confirm the assigned low hazard. The boreholes encountered groundwater at a depth of about 8 m, a meter shallower than expected. This ground water, however, was likely perched on the bedrock surface. At this ground water depth, 0.3 m to 1.4 m of the sediment overlying the conglomeratic bedrock is beneath the ground water table. Analysis of geotechnical data listed on the borehole logs (Appendix A), however, confirmed that saturated sediment are too dense (measured blow counts greater than 50) to liquefy. Thus the Priority IV classification is appropriate for this site even though ground water was slightly shallower than expected from the map by Hecker et al.(1988).

Site Specific Analyses for Liquefaction and Sensitive Clays

All bridge sites classed as possibly susceptible to liquefaction from the regional evaluations were further evaluated using site-specific procedures. These analyses required information from foundation reports, including penetration resistance, grain-size data, Atterberg limits, and stratigraphic cross sections. Where the required data were inadequate or unavailable, which was the case for nearly all pre-1960 bridges, the site was classed as Priority I if the site is near a river or other water crossing, near a steep slope or high (greater than 5 m) embankment. If remote from these features, the site was classified as Priority III, indicating insufficient information for a site specific analysis, but sufficiently low ground failure hazard to warrant a moderate to low priority for further investigation. Where adequate site information was available, the following analyses were applied.

Screening for Extra-Sensitive Clays

A phenomenon closely related to liquefaction is strength loss in sensitive clays. Sensitive clays are fine-grained soils with flocculated or "cardhouse-type" structures that may collapse with catastrophic loss of strength when disturbed by cyclic deformations during earthquakes. Flocculated structures are commonly created as fine-grained sediment that was deposited in saline lakes or seas. Because Lake Bonneville and predecessor lakes became saline as evaporation occurred in their later stages, clays with flocculated structures are possible in many of these lake bed sediments. Soil properties used to identify extra sensitive soils include low-plasticity clays with liquid limits less than 40 percent, liquidity indexes (LI) greater than 0.5, moisture contents (w) greater than 0.9 times the liquid limit, and a low penetration resistance (Youd, 1998). In the latter instance, extra sensitive clays have corrected standard penetration resistances less than 5 blows per foot (300 mm) or corrected cone penetration resistance less than 1,000 kPa.

Possibly sensitive clays were identified beneath several bridge sites near the Great Salt Lake and in the Bonneville Salt Flats. For example, the foundation investigation for the bridge at the AB line on I-80 near the Solar Salt Plant west of Salt Lake City contains the following information. Three boreholes at the site penetrated a soft clay layer between depths of 4 m and 13 m. The average corrected standard penetration blow count, $(N_1)_{60}$, in the layer was 5.0 blows per foot (300 mm). These blow counts indicate a marginal likelihood that sediments in these layers are sensitive. Liquid limits determined from three soil samples taken from these layers were 33 percent, 38 percent, and 59 percent, respectively. Plasticity indexes for the same samples were 13 percent, 16 percent, and 33 percent and the respective moisture contents were 31 percent, 39 percent and 33 percent. These data yield liquidity indexes of 0.85, 1.07, and 0.27. The soils classify as AASHTO soil types A-6 and A-7. Thus, two of these samples indicate that the clay could be sensitive. To be conservative, the site was classed as potentially sensitive, but classified as a Priority II for further investigation because of the flat terrain surrounding the site. Even if strength loss should occur in the clay, there is little potential for damaging ground displacements at this site. Additional tests to verify whether these soils are sensitive, should include strength and sensitivity tests on undisturbed samples extracted from the possibly sensitive layer. A 1-m to 2-m thick "lime sand" overlies the possibly sensitive clay layers. That sediment was determined to be liquefiable using penetration resistance procedures, further warranting the Priority II classification for the site.

Soil Classification Evaluation

Liquefaction is generally restricted to coarse-grained soils (silts, sands and gravels) that are sufficiently loose and uncemented which tend to compact during seismic shaking. In undrained or poorly drained soils, the tendency to compact leads to increased pore pressures that may ultimately generate a liquefied condition. Clay bonding between particles inhibits seismic compaction of fine-grained and cohesive coarse-grained soils, preventing compaction and pore pressure generation. Based on reported behavior of cohesive soils in areas of strong ground shaking, primarily from China, Seed and Idriss (1982) developed the criteria listed in table 4. These criteria, commonly called "the Chinese criteria," are widely used by geotechnical engineers to differentiate between liquefiable and nonliquefiable soils. Field performance during recent earthquakes indicates that the criteria in table 4 are generally conservative predictors of field performance (Gilstrap and Youd, 1998).

Table 4: Criteria for assessing liquefiability of fine-grained soils (modified from Seed and Idriss, 1982)

Criteria required for liquefaction of fine-grained soils (All three criteria must be met for soil to be liquefiable)
■ Clay Fraction (Percent Finer Than 0.005 mm) < 15% ■ Liquid Limit (LL) < 35% ■ Moisture Content (MC) > 0.9 LL

Based on the criteria listed in table 4, the following AASHTO soil types are non-liquefiable: A-2-5, A-2-6, A-5, A-6, A-7-5, and A-7-6 (Youd, 1998). For sites where soil classifications have been determined from laboratory tests on retrieved soil specimens, those classifications may be used for liquefaction hazard screening. Qualitative soil descriptions noted on borehole logs may also be used provided conservative interpretations have been made of soil types. To be conservative, soils described with clay as the principal constituent may be considered nonliquefiable (i.e., sandy clay, silty clay, soft clay, etc.). These soils likely have clay contents greater than 15 percent. All other soils, including those with inadequate soil information, should be considered liquefiable for screening purposes.

Soil types were used to classify liquefaction hazard for several bridge sites north and west of the Great Salt Lake. The bridge carrying I-84 over US 30S at station 2633+74 (west of Tremonton) was classified as nonliquefiable from soil classification information. That site is underlain by thick deposits of late Lake Bonneville sediment. Borehole logs from the bridge foundation investigation indicate that soils underlying the bridge consist of alternating layers of AASHTO types A-6, A-7-5, and A-7-6. Because these soils are too clay rich to be liquefiable, the site was classified as having low liquefaction hazard and low priority for further investigation (Priority IV). Similarly, two bridge sites on I-80 near the Utah-Nevada border were classed as low liquefaction hazard based on soil classification data. Those sites include crossings over a county road at station 68+60 and over an access road at station 130+51. The borehole logs from these sites indicate that all soils under the site are clays. Both sites were classified as low liquefaction hazard and low priority for further investigation (Priority IV).

Liquefaction Analysis Using Standard or Cone Penetration Data

The most commonly used technique for evaluating liquefaction resistance is the "simplified procedure" developed by the late Professor H. Bolton Seed and his colleagues (Seed and Idriss, 1971; 1982; Seed et al.,1985). This procedure was further updated by Youd et al. (1997). The original procedure relied on standard penetration resistance and fines content data as the key soil properties for the analysis. The procedure has been further developed for cone penetration resistance measurements (Robertson and Wride, 1997). Where adequate data is available, the simplified procedure was used to evaluate liquefaction resistance of sediments beneath bridge sites in Utah that had not been classified as low hazard (Priority IV) sites from the regional screening and soil classification evaluations.

The standard penetration test was used exclusively in Utah highway engineering practice prior to 1996. Most pre-1989 standard penetration tests were conducted with nonstandard equipment and poorly documented procedures. The quality of these older penetration tests is uncertain. These deficiencies required assumptions, such as estimated energy ratios and corrections for nonstandard sizes of split-spoon samplers. Fines content data were also seldom included on the borehole logs, requiring estimates from soil descriptions. The required assumptions were conservatively made to yield estimates of liquefaction hazard that are more likely to be over predictions than under predictions. Thus, additional field investigations are more likely to reduce the estimated hazard than increase it.

The 1997 site investigations for the reconstruction of I-15 in Salt Lake County employed cone penetration tests (CPT) as well as high-quality standard penetration tests (SPT). Fine contents were measured and reported on the borehole logs. The combination of CPT and SPT tests and along with measured soil properties allowed more confident prediction of liquefaction resistance for this segment of highway. Even so, the amount of data available at the time of the 1997 evaluation of liquefaction hazard for this section study was too sparse to fully define the liquefaction hazard beneath most of the bridge sites. Additional foundation investigations for the bridges to be constructed will likely increase the amount of data and increase the certainty of the predictions.

For each bridge site evaluated with the simplified procedure, a profile or log was developed delineating depths and thicknesses of potentially liquefiable layers. A complete description of these profiles is contained in appendix A of the project report. Figures 8, 9, and 10 illustrate the liquefaction profiles developed for the bridge sites in the I-15 corridor of Salt Lake County. These profiles show depths and thicknesses of potentially liquefiable soils for the northern, central and southern segments of the corridor, respectively. As indicated on the figures, nearly every bridge site in the corridor is underlain by one or more layers of potentially liquefiable sediment. Sites underlain by liquefiable layers with thicknesses greater than 0.3 m were

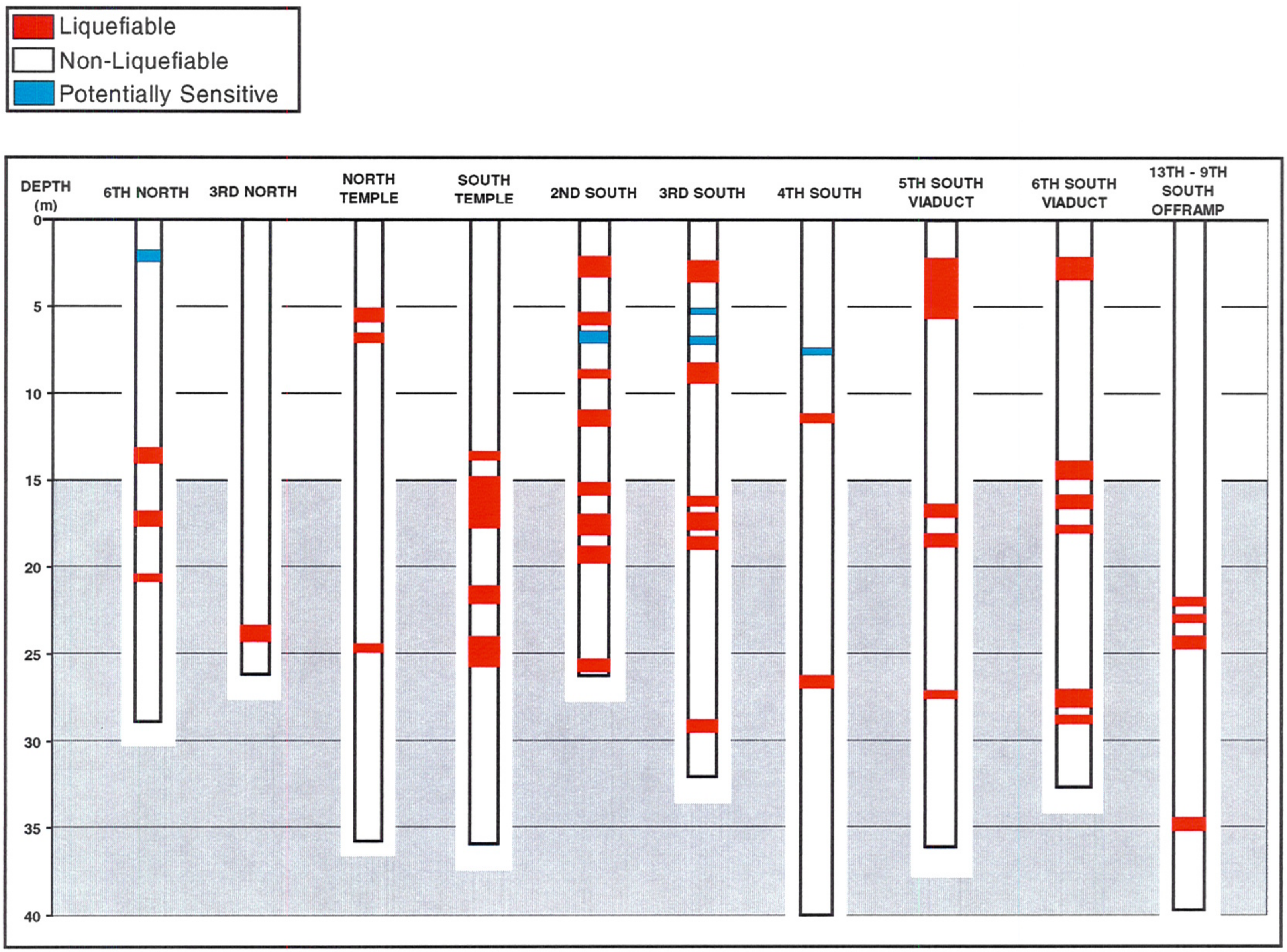

Figure 8: Logs for the I-15 corridor in Salt Lake County from 600 North to the 900 and 1300 South offramp showing potentially liquefiable layers in soil profile

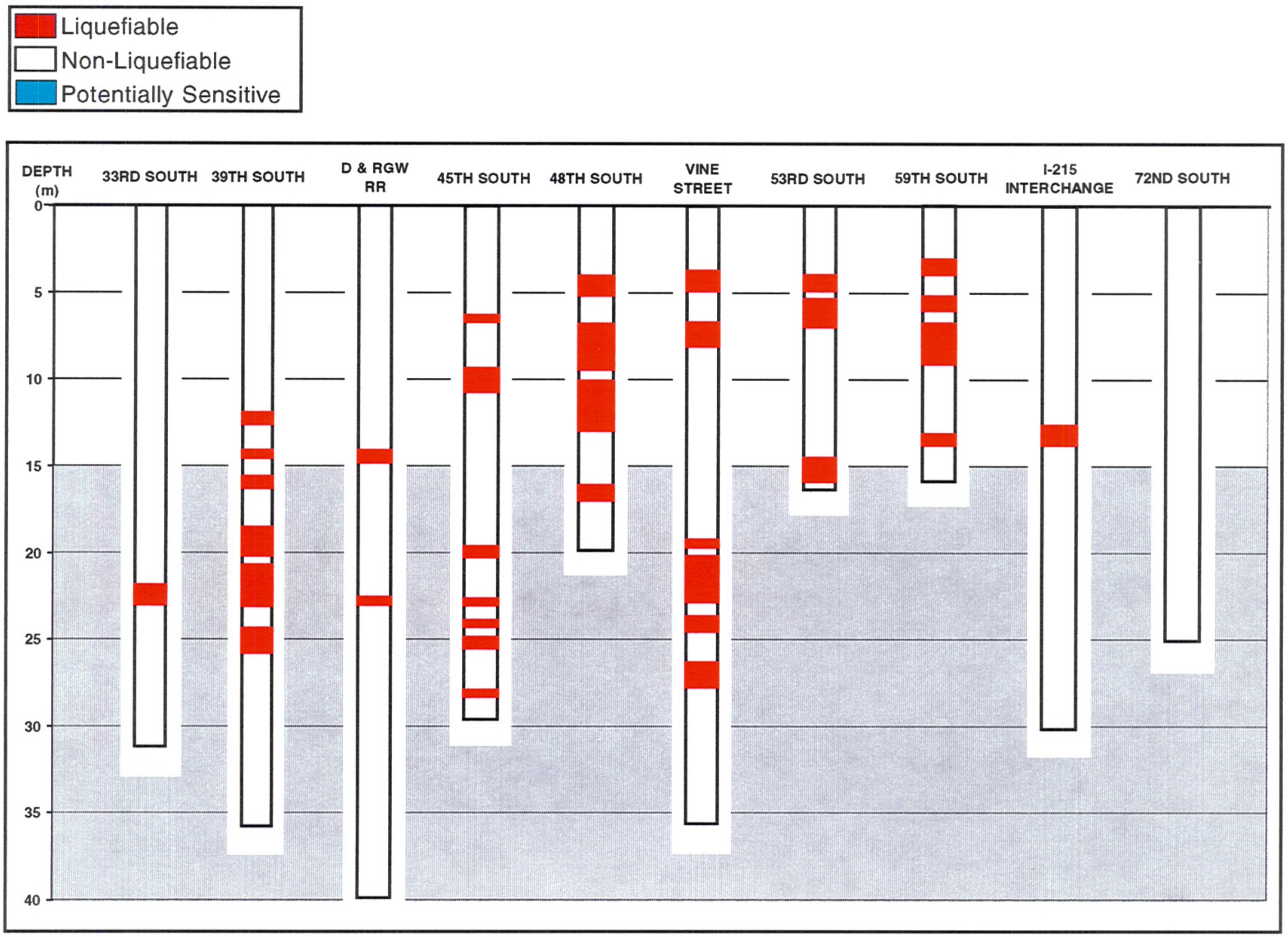

Figure 9: Logs for the I-15 corridor in Salt Lake County from 3300 South to 7200 South showing potentially liquefiable layers in soil profile.

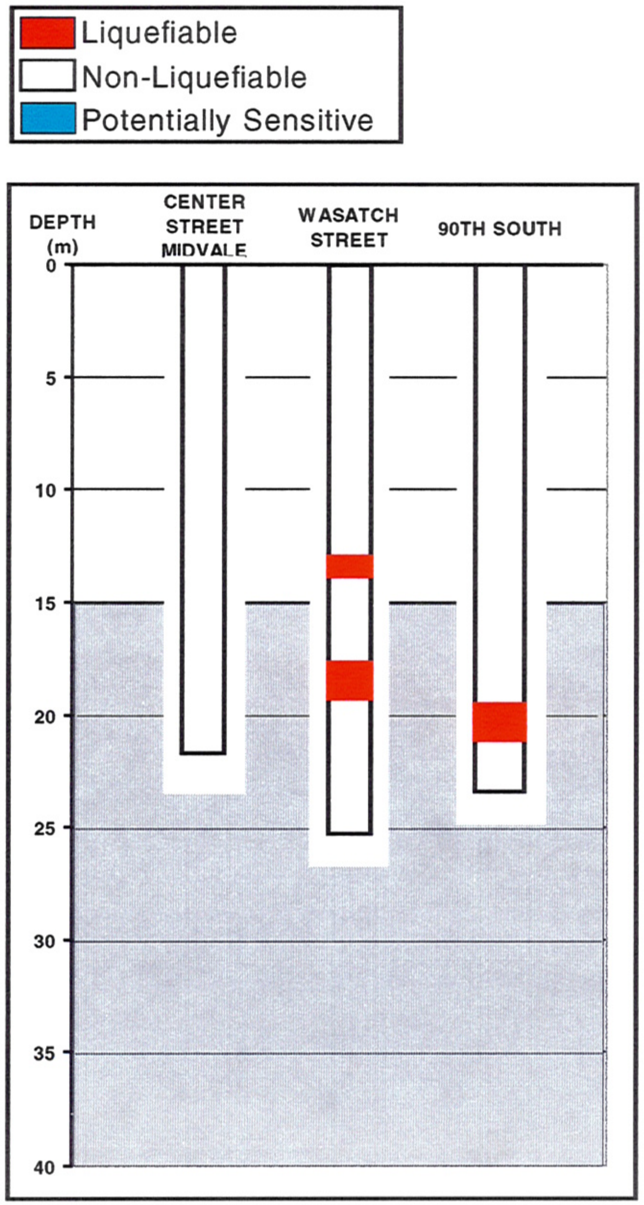

Figure 10: Logs for the I-15 corridor in Salt Lake County from Center Street in Midvale to 9000 South showing potentially liquefiable layers in soil profile.

classified as Priority II for further investigation. This classification is warranted because of the generally low ground failure potential owing to nearly flat ground along the corridor. Because of the importance of these structures, however, and the unusual opportunity to further evaluate liquefaction hazard at little additional cost as part of foundation investigations for the new bridges, a more complete assessment of liquefaction hazard should be made as part of the I-15 reconstruction effort.

Profiles showing depths and thicknesses of possibly liquefiable layers were made for each bridge site in Utah where regional investigations indicated potential for liquefiable sediments and where adequate geotechnical information was available to estimate liquefaction resistance. These profiles are used as figures in the project report. A summary listing the bridge locations and the cumulative thickness of liquefiable sediment in the upper 15 m of the soil profile is given in table 7 for all of the Priority I sites. As noted above, all of the Priority I sites are at river or creek crossings. Twenty-five bridges are listed as Priority I sites. Because of the general greater importance of interstate highway bridges, the Priority I sites were divided into two subcategories: Priority I(1) sites for bridges in the Interstate highway system (13 bridges) and Priority I(2) sites for bridges in the Federal and State Highway system (12 bridges).

Evaluation of Consequences of Liquefaction

Liquefaction of subsurface sediment layers may or may not be harmful to bridge structures depending on whether the liquefied condition induces damaging ground displacements or loss of foundation bearing strength. If predicted ground displacements or loss of bearing strength is likely to damage the bridge, mitigative measures should be implemented. Typical measures include ground modification to increase liquefaction resistance of the soil or strengthening of the structure to resist ground displacement.

For those sites where predicted displacements are too small to be damaging to the bridge and the foundation is capable of transferring loads to competent layers, liquefaction would be harmless to the bridge and of little engineering concern. In those instances, the sites were classed as Priority IV sites even though liquefaction might occur. The following analyses are required to evaluate potential for damaging ground displacement and loss of foundation bearing strength.

Embankment Stability Analysis

The first step in evaluating ground displacement hazard is to assess the stability of bridge approach embankments. If massive embankment instability were to occur, consequent ground displacements could disrupt bridge foundation elements as well as the embankment. Embankment stability can be evaluated using a standard limit equilibrium analyses where all subsurface liquefiable layers are assigned post liquefaction residual strengths. Computer programs such as *SlopeW* or *UTEXAS2* may be used for such stability analyses. If the static factor of safety against instability is greater than 1.1, the embankment can be considered safe against mass instability as a consequence of liquefaction and the analysis proceeds to the next step. If the factor of safety is 1.1 or less, the site should be classed as potentially hazardous and further site-specific investigations initiated to fully assess the hazard and recommend remedial measures.

The available geotechnical data were insufficient to fully assess embankment stability for any of the bridge sites evaluated, including the I-15 sites in Salt Lake County. The following analysis, using the best available data and conservative assumptions, demonstrates the procedure. The site chosen for the analysis is the off-ramp from southbound I-15 to eastbound 600 South in Salt Lake City. Site information and geotechnical data from nearby boreholes were used in the analysis along with slope data estimated from topographic maps. Conservative assumptions were made where data were unavailable.

The embankment is approximately 9 m high. The boreholes from which the soil information was estimated, holes DH-15 and DH-15A, are located approximately 50 m from the off ramp embankment. Gerber (1995) compiled extensive borehole information for the site including static strength values for various sediment layers. These strengths were used in the analysis, except for the liquefiable layers for which we assigned estimated post-liquefaction residual strengths. Application of the simplified procedure indicates that liquefiable layers lie between depths of 1.2 m to 4.0 m and 17.1 m to 17.7 m. The deeper layer was ignored because it is too deep to adversely affect slope stability. A residual strength of 28 kPa was estimated for the upper liquefiable layer based on a corrected blow count of 16 and charts published by Seed and Harder (1990) (figure 11). The limit-equilibrium analysis was applied with the aid of the computer program *UTEXAS2.* The result was a factor of safety, FS, of 1.46 against static failure

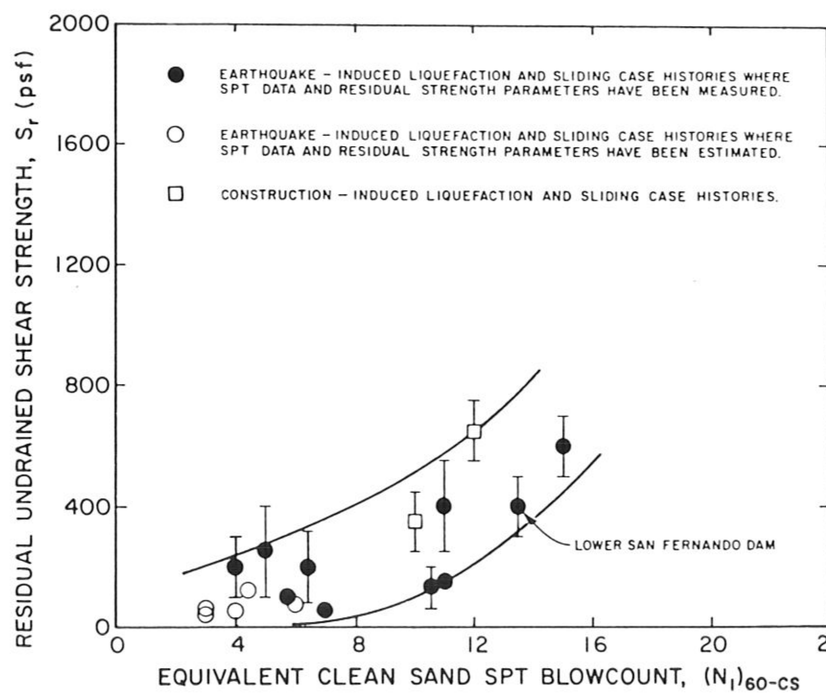

Figure 11: Empirical relationship between residual shear strength and $(N_1)_{60cs}$ (After Seed and Harder, 1990)

for the embankment. A cross section of the embankment along with the critical failure surface determined from the analysis is plotted on figure 12.

With a factor of safety of 1.5, this embankment was classed as safe against mass instability, even if liquefaction were to occur. Deformation of the embankment due to strong ground shaking, however, still could cause damage to the bridge. Thus, the analysis proceeded to the next step as indicated in the flow chart reproduced in figure 2.

Embankment Deformation Analysis

Damaging ground deformations may occur within or beneath embankments or slopes as a consequence of liquefaction, even though the site may be stable against flow failure. Such deformations occur as a consequence of soil softening and yielding due to liquefaction and inertial forces generated during the earthquake. In these instances, cyclic mobility and limited strains within liquefied layers may lead to detrimental ground deformations and displacements. Analysis of embankment deformation at liquefiable sites is complicated because of the complex nature of constitutive relations for liquefied soils. In particular, stress-strain relations are very complex for moderately dense or dilative soils that may deform under either undrained or partially drained conditions. Embankment deformation can be estimated using mechanistic (Newmark sliding block), finite element, or other numeric analyses. These analyses usually neglect the restraining influence of the bridge structure, which is difficult to quantify.

Youd (1998) suggests the following simplified screening criteria for dynamic displacements based on the mechanistic analyses. A key parameter in the mechanistic analysis is the yield acceleration, which is defined as the pseudostatic horizontal acceleration required to reduce the calculated static factor of safety to 1.0. For this analysis, all liquefiable layers are assigned the appropriate residual strength as in the static stability analysis noted above. During earthquake shaking, only acceleration pulses with amplitudes greater than the yield acceleration generate permanent slope displacement. Displacement continues only as long as dynamic inertial forces exceed the resisting forces (factor of safety transiently less than 1.0). Past analyses show that the amount of dynamic displacement decreases markedly as the static factor of safety, or yield acceleration increases (Makdisi and Seed, 1978). These analyses indicate that permanent displacements are generally small for sites with an adequate static factor of safety. Youd (1998) suggests that acceptable displacements (less than 100 mm) are likely with the combinations of earthquake magnitudes and factors safety listed in table 5.

Table 5: Combinations of earthquake magnitudes and factors of safety that should yield ground deformations less than 100 mm

Earthquake Magnitude	Factor of Safety
6.5	1.5
7.5	2.0
8.5	2.5

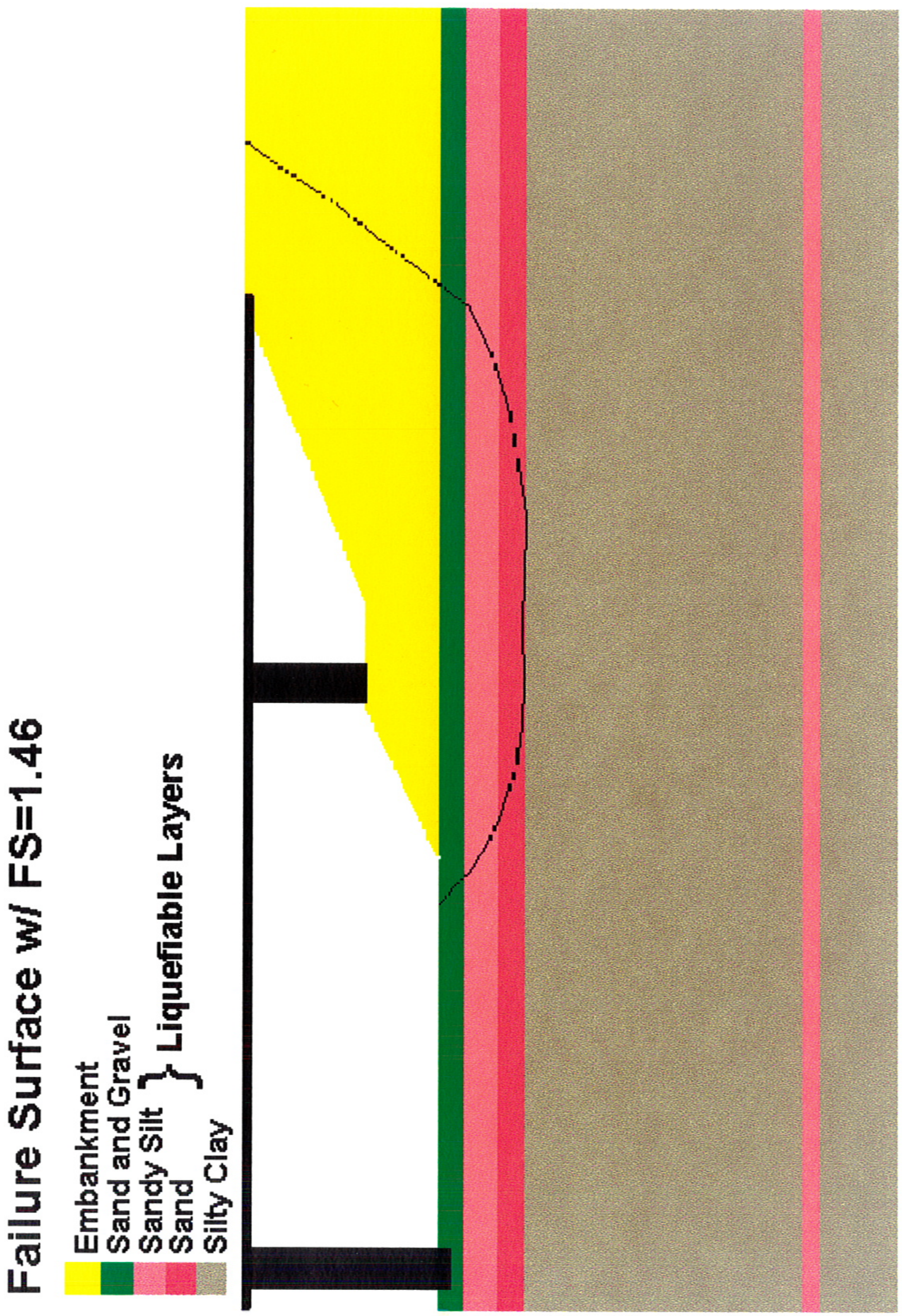

Figure 12: Profile of 600 South bridge site showing critical failure plane with factor of safety, FS, of 1.46

These criteria were applied to the 600 South off ramp of I-15. The calculated factor of safety for the embankment is approximately 1.5 as noted above. A maximum earthquake magnitude of about 7.0 to 7.3 is generally estimated for the Salt Lake segment of the Wasatch fault (Arabasz et al., 1992). For this magnitude and factor of safety, the criteria above indicate that deformations at the 600 South offramp could exceed 100 mm.

To further estimate the dynamic deformation at the 600 South offramp, a second estimate was made using the finite element analysis program QUAD4M. Input data required for this analysis included the information used to calculate the static factor of safety, a shear-wave velocity profile, and a time history of accelerations expected during a magnitude 7.0 earthquake. Using this information, the Quad4M analysis calculated an acceleration time-history at various points in the embankment. All accelerations that exceeded the yield acceleration (approximately 0.1 g for the 600 South site) on the critical failure surface shown in figure 12 were double integrated with respect to time to estimate permanent deformation. The estimated deformation from this analysis was less than 40 mm.

Most modern reinforced-concrete, steel, or heavy timber bridges should be able to withstand 100 mm to 200 mm of unrestrained embankment deformation without significant damage. Some lightweight timber bridges have been damaged by displacements smaller than 100 mm, but these types of bridges are not commonly used for highway bridges in Utah. Such small ground displacements are usually accommodated by compression or shear of the displaced soil rather than structural deformation. The 40 mm of displacement predicted for the 600 South off ramp is smaller than the 100 mm to 200 mm of allowable displacement. Thus, this structure was classed as not susceptible to damage from embankment deformation. Because of inadequate local site data, this analysis was conducted as a demonstration. This and other structures should be reanalyzed as more geotechnical data are compiled during foundation investigations for the new bridges to be built in the I-15 corridor.

Lateral Spread Analysis

As noted in the introductory paragraph, most liquefaction-induced bridge damage has been due to lateral spread of flood plain deposits toward river channels. Lateral spread displacement is generally estimated using empirical equations such as those developed by Bartlett and Youd (1995). Data required for lateral displacement analyses include the stratigraphic and penetration data used for evaluation of liquefaction resistance, grain-size distribution data, and site topography. If predicted lateral spread displacements are tolerable (100 mm or less for most highway bridges) the site may be classed as not susceptible to damage from lateral spread, and the analysis proceeds to the next step. If predicted displacements are potentially damaging (typically greater than 100 mm), a more detailed site study should be initiated to more accurately define lateral spread potential and to develop remedial measures where predicted displacements are sufficiently large to damage the bridge.

The empirical equations developed by Bartlett and Youd (1995) were used to calculate lateral spread displacements for this study. For mildly sloping ground conditions, such as those at the 600 South off ramp in Salt lake City, the following equation is recommended by Bartlett and Youd:

$$\text{LOG } D_H = -15.7870 + 1.1782\text{ M} - 0.9275\text{ LOG R} - 0.0133\text{ R} + 0.4293\text{ LOG S} + 0.3483\text{ LOG } T_{15} + 4.5270\text{ LOG }(100 - F_{15}) - 0.9224\text{ D}50_{15} \qquad (1)$$

where: D_H is the estimated lateral ground displacement in meters; M is the earthquake magnitude (moment magnitude); R is the horizontal distance from the site to seismic energy source, in kilometers; S is the ground slope, in percent; T_{15} is the cumulative thickness of saturated granular layers with corrected blow counts less that 15, $[(N1)_{60} < 15]$, in meters; F_{15} is the average fines content (fraction of soil sample passing a No. 200 sieve) for granular layers included in T_{15}, in percent; $D50_{15}$ is the average mean grain size, in mm, for granular layers included in $T_{15.}$

The only layer which poses a significant lateral spread hazard is the liquefiable layer between depths of 1.2 m and 2.7 m. The lower liquefiable layers are too thin and too deep to pose a lateral spread hazard (depth > 15 m). The following input values for the lateral spread analysis were estimated for the 600 South offramp: M = 7.0, R = 3.0 km (under the 5.0 km limit), S = 0.25 percent, T_{15} = 1.5 m, F_{15} = 75 percent (over the 50 percent limit), and $D50_{15}$ = 0.06 mm (under the 0.1 mm limit). Entering these values into equation (1) yields a predicted displacement of 11 mm. Unfortunately, many of the required values are outside the limits specified by Bartlett and Youd (1995), which increases the uncertainty of the predicted displacement. The analysis was repeated with site variables outside the recommended limits adjusted to the nearest acceptable limiting value; that is R= 5 km rather than 3 km; F_{15} = 50 percent rather than 75 percent; and $D50_{15}$ = 0.1 mm rather than 0.06 mm. The estimated displacement from this second analysis is 140 mm. The actual displacement is likely to lie between 11 mm and 140 mm. Even if 140 mm of ground displacement were to occur, a well-built highway bridge should withstand the displacement with minimal damage. Thus, possible lateral spread displacement at this site was sufficiently small to pose a large hazard and the investigation proceeded to the next step as shown on the flow chart in figure 2.

Ground Settlement Analysis

If earthquake-generated embankment deformation and lateral spread displacements are tolerable, the next step is to estimate the expected ground settlement. Settlement occurs as a consequence of compaction of cohesionless soils when subjected to earthquake shaking. Empirical procedures developed by Tokimatsu and Seed (1987) were applied to predict ground settlement.

The premise of the Tokimatsu and Seed procedure is that earthquake shaking generates cyclic shear strains that compact granular soils, causing volumetric strain. Where drainage cannot occur rapidly, the tendency to compact also generates transient pore water pressures that prevent immediate decrease in volume. However, as pore pressures dissipate, the layer consolidates, producing volumetric strain and ground settlement. The induced volumetric strains are primarily a function of the amplitude of the cyclic shear strains generated by the earthquake and the initial relative density of the sand. The cyclic shear strains are a function of the cyclic stress ratio (CSR), relative density, and earthquake magnitude. Tokimatsu and Seed corrected CSR for magnitude by dividing by the appropriate magnitude scaling factor (Youd et al., 1997). Relative density was estimated directly from corrected penetration resistance, $(N_1)_{60}$. For silty sands, $(N_1)_{60}$ was

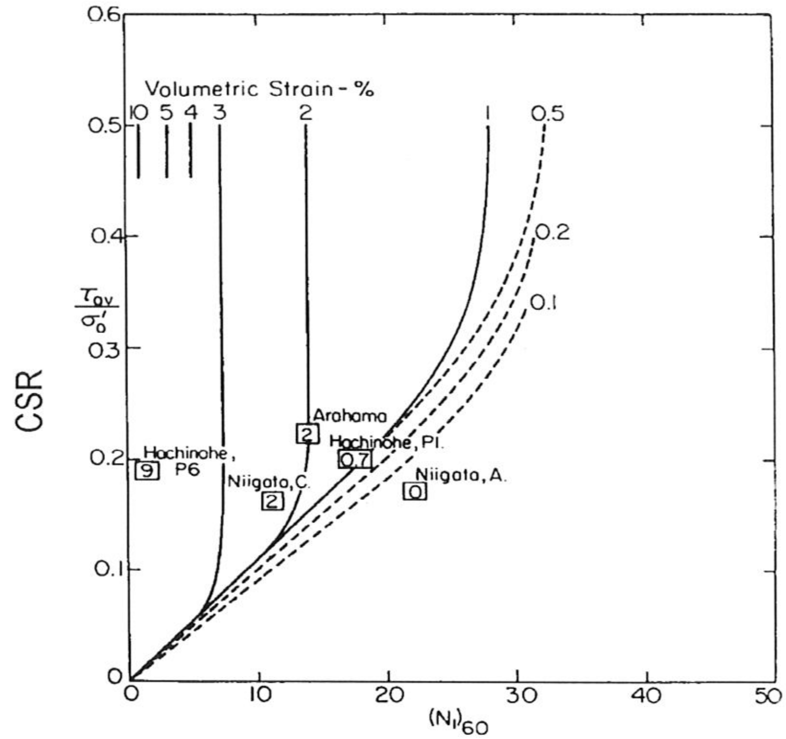

Figure 13: Curves for estimating volumetric strain at liquefiable sites (After Tokimatsu and Seed, 1987)

corrected to $(N_1)_{60cs}$ using the correction factors specified for calculation of liquefaction resistance. Figure 13 is a synthesis diagram developed by Tokimatsu and Seed from available laboratory test data and field observations of earthquake-induced settlements in clean sands. The volumetric strain is then multiplied by the layer thickness, assuming one-dimensional consolidation, to compute the change of layer thickness. The changes of thickness from all layers at the site are then summed to estimate ground settlement.

Settlements were calculated for the 600 South off ramp site using this procedure. Volumetric strains were calculated for the cohesionless layers between depths of 1.2 m and 2.7 m. The values of $(N_1)_{60}$ and the CSR were taken from the analysis of liquefaction resistance. Table 6 tabulates the fines-content corrections, estimated volumetric strains, and settlements.

Based on past experience, Youd (1998) suggests bridges on shallow foundations should not be damaged by settlements of 25 mm or less and that well built bridges on deep foundations can withstand 100 mm or more settlement without damage. Because the bridges in the 600 South interchange are all constructed on deep foundations, a general ground settlement of 34 mm should not adversely affect these structures. Thus, this site was classified as low hazard for ground settlement.

Table 6: Values used in settlement calculations for 600 South southbound off-ramp from I-15. Layers are at depths of 1.2 m and 2.7 m

Layer Thickness	$(N_1)_{60}$	$(N_1)_{60\text{-cor}}$	CSR	Volumetric Strain (%)	Incremental Settlement
1.5 m	11	19	0.476	1.5	23 mm
1.2 m	19	28	0.524	0.9	11 mm
Total Settlement					34 mm

Bearing Capacity Analysis

If liquefaction-induced ground deformations and ground settlements are tolerable, the remaining possible liquefaction-induced hazard is loss of foundation bearing strength. Loss of bearing strength could lead to penetration of shallow or deep foundations into the liquefied sediment or possibly to buckling of piles as a consequence of reduced lateral resistance in liquefied soil layers.

A standard bearing capacity analysis is used to assess bearing capacity for shallow foundations. Residual strengths are assigned to all the liquefiable layers, however, for the analysis. For axial load capacity of deep foundations, standard practice is to conduct a load capacity analysis with negligible strength assigned to liquefiable layers. Lateral load resistance of deep foundations is usually estimated by multiplying the lateral resistance for nonliquefied conditions by a factor ranging from 0.1 to 0.3, depending on relative density, to account for degradation of lateral support due to liquefaction.

If the load capacity analyses indicate an adequate factor of safety (say 1.5 or greater) against loss of foundation failure, the site may be classed as nonhazardous and immune to detrimental effects of liquefaction (Priority IV site) even though liquefaction of some subsurface layers may occur. At this juncture, all of the possible detrimental effects of liquefaction have been considered and determined to be nondamaging to the bridge. Conversely, if the load capacity analysis indicates a marginal factor of safety (less than 1.5), unacceptable foundation penetration may occur and the site is classified as a Priority I site and recommended for additional investigation and possible remediation.

Bridges near the 600 South offramp of I-15 are founded on deep pile foundations which likely extend to deep competent layers. Foundation plans were not available for this study, however, so an analysis of load capacity could not be made. The soils beneath the 600 South site contain liquefiable layers at depths of 1.2 m to 4.0 m, 17.1 m to17.7 m, 24.4 m to 25.6 m, 26.5 m to 27.7 m, and 28.6 m to 31.1 m. If these layers provide a substantial part of the bearing resistance, failure in the form of pile penetration or excessive lateral displacement might occur during an earthquake. For future structures at the site, care should be taken to assure that liquefiable layers are not relied on to provide significant bearing strength.

Prioritization of Bridge Sites for Further Investigation

The primary objective of this study was to prioritize the bridge sites in Utah for further investigation. From the various evaluations noted above, bridge sites were prioritized into the four categories listed in the Introduction. A few hundred bridges were considered in the regional liquefaction hazard investigations, and approximately 325 bridge sites were reviewed and analyzed using the site-specific steps in the screening guide. Of these sites, about 279 were identified as underlain by possibly liquefiable soil layers. Twenty-five of these sites were identified as Priority I sites for further investigation (table 7).

The Priority I sites are at river crossings, a setting in which most past bridge damage due to liquefaction has occurred. These sites were further prioritized into two subcategories depending on importance. Bridges in the Interstate highway system were given higher priority (Priority I(1)) based on importance of the structure to the transportation system. Bridges in the Federal and State highway systems were given a slightly lower priority (Priority I(2)) because of the lower amount of traffic and the general availability of alternate routes that could be used if bridge damage should obstruct traffic operations. In general, bridges on highways without readily available bypass routes are given higher priority for investigation and mitigation because of the lack of an alternate route (Buckle and Friedland, 1995).

The remaining 254 bridge sites of the 279 sites that were possibly underlain by liquefiable sediment were classed as Priority II and III sites. (Priority II sites have confirmed liquefiable layers in the foundation, while the foundation information is insufficient for Priority III sites to assess liquefaction resistance of subsurface layers. The information for sites with either priority was insufficient to assess the consequent ground failure hazard.) The Priority II and III sites are tabulated in appendix A of the project report. These sites should be immune to damaging lateral ground displacements, but could be damaged by either liquefaction-induced ground settlement or loss of foundation bearing strength.

A principal emphasis of this screening evaluation was on the I-15 corridor in Salt Lake County. Two investigations were made of bridge sites in this corridor - a preliminary study using older and generally lower quality data from 1950 and 1960 foundation investigations for the original highway construction, and a follow up study using higher quality data from reconnaissance investigations for planned reconstruction. Both studies show that nearly all of the I-15 bridge sites in Salt Lake County are underlain by possibly liquefiable sediment. Use of the higher quality data reduced uncertainty in the evaluation and reduced the amount of sediment classified as liquefiable to about half that of the preliminary study. The higher quality study identified possibly liquefiable layers in the upper 15 m of sediment, beneath 24 of the 33 bridge sites evaluated in the I-15 corridor. The cumulative thicknesses of these liquefiable layers range from 1 m to 7 m. These sites are classed as priority II sites for future study because they have a confirmed presence of possibly liquefiable sediments but they are generally flat and not at river crossings or near other bodies of water. Nevertheless, because of the critical importance of the I-15 corridor and the opportunity to more fully evaluate the liquefaction hazard as part of the design of the new bridges, the liquefaction hazard should be fully evaluated as part of the reconstruction project.

Table 7: Priority I bridge sites in Utah (highest priority for further investigation) along with assessed thickness of liquefiable layers at those sites

Bridge Location	Borehole Number	Depth Intervals of Liquefiable Layers (m)	Priority
I-15 over Malad River	6	G.W.T.-6.71, 8.23-9.75, 19.2-25.6	I(1)
I-84 / I-15 Over Malad River	1	G.W.T.-3.35, 7.62-14.02, 15.54-17.98, 19.51-26.21, 36.27-37.8, 39.32-45.72	I(1)
I-15 Over Bear River	5	2.44-7.32, 12.5-14.02, 15.24-16.76	I(1)
I-15 Over Weber River	2	G.W.T.-2.13, 29.57-35.51	I(1)
I-84 Over Cottonwood Creek	2	7.01-11.3, 15.2-15.85	I(1)
I-80 Over Jordan River	A-3	G.W.T.-3.05, 10.06-12.8, 17.07-18.9	I(1)
I-215 Over Jordan River (North)	S-17	5.5-7.2, 7.9-10.6, 21.65-23.4	I(1)
I-215 Over Jordan River (South)	3	1.98-3.05	I(1)
I-15 Over Spanish Fork River	U68	3.66-12.19, 12.8-16.76, 17.07-24.08	I(1)
I-15 Over Sevier River	3	G.W.T.-3.66	I(1)
I-70 Over Sevier River	6	1.52-1.98	I(1)
I-15 Over Santa Clara River	U5	G.W.T.-4.57, 7.47-10.67	I(1)
I-15 Over Virgin River	U3	G.W.T.-8.84, 9.75-16.15	I(1)
HWY 6 Over Spanish Fork River	insufficient information		I(2)
HWY 89/91 Over Logan River	insufficient information		I(2)
HWY 89/ 91 Over Little Bear River	DH3	1.52-3.05, 9.45-11.28, 21.34-23.47	I(2)
HWY 89/91 Over Ogden River	1	5.49-6.71, 8.99-10.06, 16-21.03, 31.7-33.53	I(2)
HWY 89/91 Over Provo River	1	7.16-16.76	I(2)
HWY 89 Over Spanish Fork River	2	1.22-5.18, 9.14-11.28	I(2)
HWY 89 Over Clear Creek	1	G.W.T.-4.42, 7.32-8.53	I(2)
HWY 89 Over Sevier River near SR-62 JCT	2	G.W.T.-2.53	I(2)
HWY 89 Over Hog Creek	1	7.62-12.59, 13.56-14.94	I(2)
SR-30 Over Malad River	1	32-34.75, 36.27-37.03	I(2)
SR-30 Over Little Bear River	2	2.13-5.49, 31.7-33.22	I(2)
2100 S (HWY 201) Over Jordan River	1	G.W.T.-6.1, 9.45-11.89, 12.95-16.76, 20.73-24.08	I(2)
G.W.T. = groundwater table			

Implementation Plan

Prioritization

Of the few hundred bridge sites considered, about 279 bridge sites were identified as underlain by possibly liquefiable layers. These 279 sites were prioritized for further investigation as follows:

Priority I sites:

Twenty-five bridge sites were classified as Priority I sites. These sites have the highest priority for further investigation and possible mitigation. The classification criteria for Priority I sites are that the sites are underlain by confirmed or possible layers of liquefiable sediment or sensitive clay that could induce damaging ground or foundation displacements during an earthquake. Liquefiable sediments were either confirmed by borehole data from past investigations or the available information is insufficient to eliminate the possibility of liquefiable sediment. In the latter instances, borehole data is insufficient to fully evaluate the liquefaction hazard, but hydrologic and geologic conditions indicate favorable conditions for the occurrence of liquefiable sediment. A second criterion for Priority I sites is that the bridge lies in an area vulnerable to ground failure, such as at a river crossing, near a river or lake or other body of water, in an area with a ground slope greater than one percent, or adjacent to a steep slope or a high embankment (greater than 5 m).

Priority II sites:

Two hundred fifty-four bridge sites were classified as Priority II sites. These sites have the second highest priority for further investigation and possible mitigation. The classification criteria for Priority II sites are that possibly liquefiable sediment was confirmed from an analysis of borehole data, but the sites are located in areas with low potential for ground failure. That is, Priority II sites are located on flat or very gently sloping terrain (less than one percent slope), more than 0.5 km away from rivers, lakes or other bodies of water, and not adjacent to steep slopes or high embankments. Liquefaction at these sites could cause ground settlement and possible foundation instability, but damaging lateral ground displacements are unlikely.

Priority III sites:

Only seventeen sites were classified as Priority III sites. These sites have the third highest priority for further investigation. Criteria for Priority III sites are the same as for Priority II sites, except that available borehole data are insufficient to fully evaluate hazard susceptibility. The sites are also located away from rivers, other bodies of water, steep slopes, or thick embankments overlying soft sediment. Liquefaction at these sites could cause ground settlement and possible foundation instability, but damaging lateral ground displacements are unlikely. These sites are assigned a moderate to low priority for further investigation.

Priority IV sites:

A few hundred bridge sites were classified as Priority IV sites. The number of bridges classified in this category was not precisely counted. These sites have the lowest priority for further investigation. Criteria for Priority IV sites are that the screening evaluation indicated that liquefiable sediments are not present at the site or are very unlikely. Most Priority IV sites are in nonliquefiable areas due to a deep water table, dense or lithified foundation materials, or very low seismic potential. Other Priority IV sites are at localities where analyses of borehole data indicate that damaging ground displacements or loss of foundation support is unlikely even though liquefaction of subsurface sediment might occur. Because of the low hazard, these bridges were assigned a low priority for further investigation or mitigation. Nevertheless, a reevaluation of the liquefaction hazard at these sites should occur whenever bridges are replaced or significant new foundation information is developed.

Implementation

To assure that bridges in Utah are safe against the deleterious effects of liquefaction, additional investigations and analyses are required. The above classification of bridge sites provides guidance for prioritization of future investigations. Based on that prioritization, the phased investigations outlined below should be implemented to mitigate the liquefaction hazard to bridges in Utah. The investigations should incorporate the following steps:

The first step in the investigation is to compile sufficient site and soil property information for accurate analyses of liquefaction resistance at bridge sites, and to eliminate the need for extrapolation or estimation of critical site or soil properties, as was necessary at many sites for the screening evaluation herein. To provide this information, the field and laboratory investigation should include the following elements:

- Sufficient cone penetration test (CPT) soundings should be made to create a complete lithologic cross-section for each bridge site. The soundings should extend to depths of 15 m or until Pleistocene or older sediment or rock is encountered. The CPT data should be used to define site stratigraphy, assign soil behavior types, and estimate soil liquefaction resistance. Use of CPT followed by borings with SPT and laboratory testing is the preferred procedure. If CPT equipment is not available, an increased number of borings and SPT may be substituted to provide the required delineation and characterization of subsurface soil layers.

- From the stratigraphic cross section, an optimal number of boreholes should be planned and drilled for standard penetration tests (SPT) and for retrieval of samples for laboratory testing. Sufficient samples should be tested to assign well-defined soil property values to each soil layer. For liquefaction hazard evaluations, those properties should include soil type, fines content, mean grain size, Atterberg limits, natural moisture content, and penetration resistance.

- Depth to water table should be measured at each site. For major bridge sites, seasonal ground water level changes should also be determined through long-term monitoring of water levels in cased wells.

- Surveys should be made to compile accurate topographic information, including embankment configuration, ground slopes, flood plain configurations, channel cross sections, etc.

The second step is to analyze the liquefaction resistance of each soil layer. The standard for this analysis is the "simplified procedure" as updated in the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (Youd and Idriss, 1997). The essentials of that procedure is outlined in appendix B of the project report. From this analysis, depths, thicknesses, and extents of liquefiable layers can be noted on the stratigraphic cross section for the site. Sensitive clay layers should also be noted if any are defined.

The final step is an evaluation of ground failure potential and foundation stability. These evaluations should include embankment stability, embankment deformation, lateral spread displacement, ground settlement, and foundation capacity. Procedures for these analysis are contained in the project report and in FHWA manuals. If the analyses indicate that one or more consequences of liquefaction are potentially damaging to the bridge, mitigative measures, such as strengthening of the foundation or stabilization of liquefiable soils, should be designed to alleviate the hazard. Conversely, if ground deformations or bearing capacities are not adversely affected, the bridge may be classed as safe even though some sediments may harmlessly liquefy during large earthquakes.

Implementation Phase I

Although most of the bridge sites in the I-15 corridor of Salt Lake County are classed as Priority II sites, the importance of these bridges and the opportunity provided by the reconstruction of I-15 increases the urgency to fully evaluate the liquefaction hazard at these sites. Foundation investigations at all bridge sites will be made as part of the reconstruction effort. The data to complete a thorough liquefaction hazard analysis can easily be collected during these investigations with little additional effort. These data should be collected and analyzed in accordance with the state-of-the-art procedures as listed in appendix B of the project report or in Youd and Idriss (1997).

Phase II

Twenty-five priority I bridge sites were identified. These sites are likely to be underlain by liquefiable deposits and are located at river crossings or other areas where damaging embankment, lateral spread or foundation displacements are most likely to occur. These sites should be given high priority for further investigation. Thirteen of these sites are in the Interstate Highway system (outside the I-15 corridor in Salt Lake County). Because of the generally greater importance of Interstate Highway bridges, these sites should be investigated before the additional 12 Priority I sites identified in the Federal and State Highway systems. Additional Priority I sites may exist at river and creek crossings in the county and city road systems; these sites were not considered in this investigation because of their general lesser importance and the lack of available foundation reports for most of these sites.

Other criteria might also be considered in prioritizing investigations, including the strength, brittleness and age of the bridge; the availability of alternate routes; and the importance of the bridge to national defense and emergency transportation systems. These additional criteria might be considered by State and local officials as they schedule the needed investigations.

Phase III

The likely liquefaction hazard at Priority II and Priority III sites is sufficiently low that a special program may not be required to investigate the hazard at these sites. Some very critical structures, such as major interchanges or bridges in essential emergency transportation routes, should be scheduled for investigation with or immediately following the Phase II investigation. Otherwise, the hazard should be investigated whenever opportunity arises, such as when bridge upgrades or replacements are planned.

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